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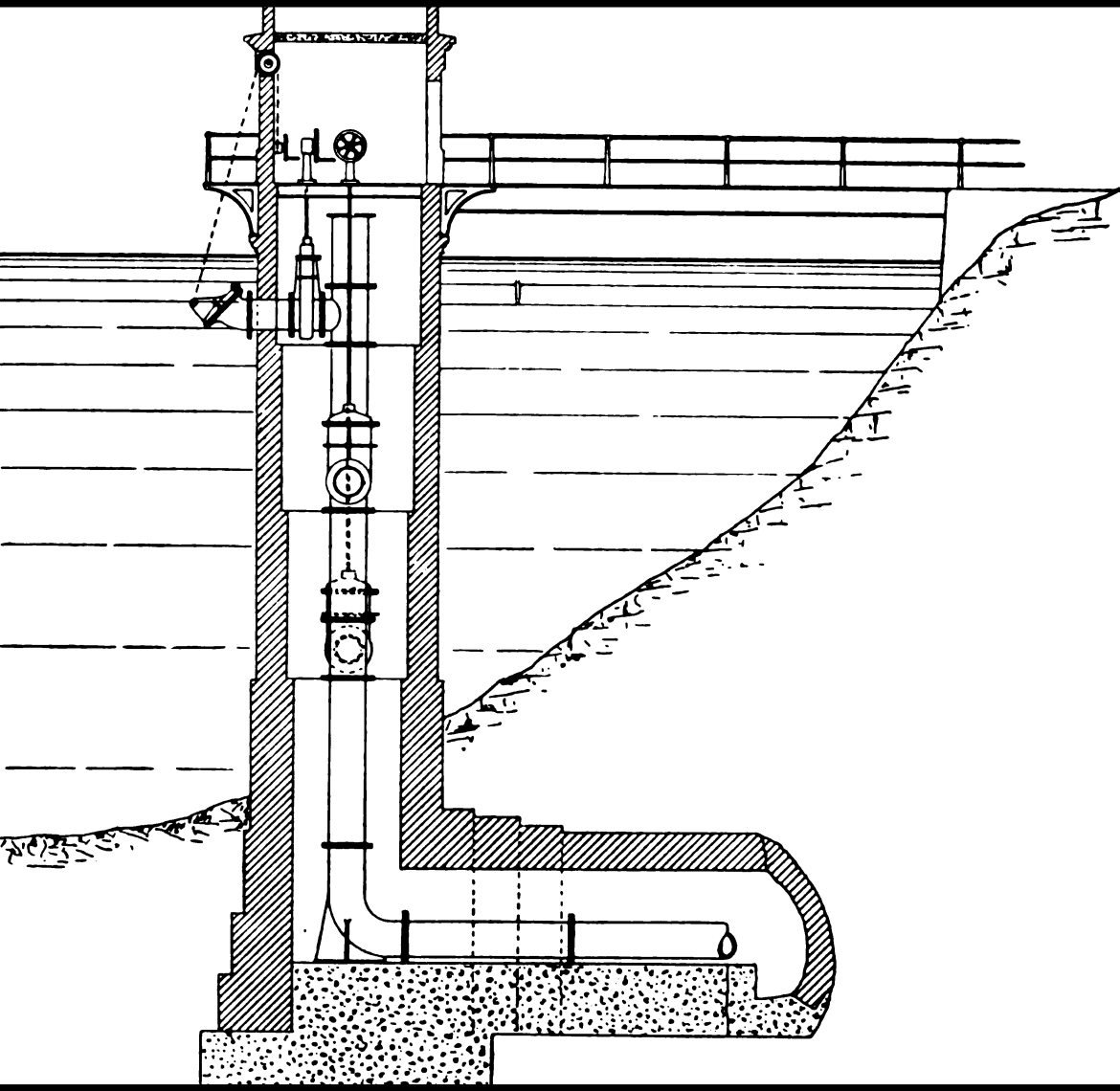

Water supply

Reginald Empson Middleton

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WATER SUPPLY.

A STUDENT'S HANDBOOK

ON

THE CONDITIONS GOVERNING THE SELECTION OF SOURCES AND THE DISTRIBUTION OF WATER.

BY

REGINALD E. MIDDLETON,
M. INST. C. E., M. INST. MECH. E., F. S. I.

WITH NUMEROUS ILLUSTRATIONS, PLANS AND TABLES.

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PREFACE.

In view of the many excellent treatises on Waterworks Engineering already in existence, it may appear that there is little opening for another work on the same subject. In the opinion of the writer and the publishers of this book, however, there is still need for a small work, more particularly suitable, perhaps, for engineering students, setting forth in a compact manner the general scientific principles on which the subject is based, and serving as an introduction to the larger, more technical, and more highly priced treatises.

It is in the hope of supplying such a need that the book makes its appearance.

Special prominence has therefore been given to such questions as the quality of the water, the interpretation of analyses, the stability of masonry dams, the flow of water through the pipes, and the general application of mathematics to the subject, etc.

At the same time, it has been the aim of the writer to bear well in mind the fact that theory is useless without practical knowledge to supplement it. Many 'rules of thumb' and practical hints, as well as descriptions of existing works, are given, based either on the writer's own experience or on that of others, and it is hoped that the young engineer will find the book useful as an introduction either to his practical work or to his study of more comprehensive books.

Little has been said about pumping machinery in dealing with supplies requiring to be artificially raised. It was felt that to have dealt with this part of the subject adequately would have unduly extended the book to little purpose, and that there are many excellent works dealing with it; it was thought, however, that the very few general observations which have been introduced might serve as a guide to a more extended examination.

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It is hoped that in all cases where other treatises have been specially consulted, suitable acknowledgment has been made in the text.

In conclusion, the writer wishes to express his great indebtedness to Mr M. T. Ormsby, M.Inst.C.E.I., Demonstrator in Municipal Engineering at University College, and to his assistant, Mr M. Mawson, Assoc. M.I.C.E., for the very valuable assistance which they have afforded him in the preparation of the book.

R. E. M.

WESTMINSTER, *June* 1903.

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WATER SUPPLY

CHAPTER I

INTRODUCTION

It is intended, in this chapter, to enumerate very briefly the different points to be considered in arranging and elaborating a scheme of water supply for a city, town or district, giving, in a general manner, the conditions which have to be taken into account at each step. In later chapters these steps will be considered separately, and treated in greater detail.

Selection of Source.—The first step of all is to determine which of the available sources of supply shall be selected—a determination governed by considerations as to the suitability of the water afforded for the purpose for which it is to be used, the sufficiency of the supply, and its cost.

Speaking generally, water may be obtained from springs, streams, rivers or lakes, and from wells. Rain water collected from the roofs of buildings and stored may be, and sometimes is, used abroad, and less frequently in this country, as a source of supply, but the first washings from roofs are polluted and should be rejected, and the use of water collected from roofs can only be justified on the plea of necessity.

In selecting a source, the first determining factor is whether the quantity of water obtainable is sufficient to afford the requisite supply, calculated at an estimated number of gallons per diem per head of population to be supplied, the estimate varying with conditions to be considered later.

The supply obtainable is estimated from the average rainfall (after making certain deductions for evaporation and seasonal changes), and from the area contributing to the supply on which the rain falls, a figure which is fairly calculable when the water to be collected is

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above ground, but which is by no means so easy to calculate when the supply is drawn from wells or springs. In order to test the accuracy of the calculations, it is desirable that the streams should be gauged for as lengthened a period as is practicable, and the products compared with the rainfall collected in one or more rain gauges, which should be again compared with a gauge which has been observed for several years, preferably thirty-six or more.

It having been decided that the supply is sufficient, its probable variations should be considered. Rivers, streams, lakes, springs and wells are all subject, in a greater or less degree, to seasonal changes, but in the case of a deep well which has been tested by pumping, for, say, three weeks, to double the extent of the water required to be drawn from it regularly on an average, without having been pumped to the bottom, and on the cessation of pumping has shown that the water gained its original level within a week, it may be assumed, with a fair degree of certainty, especially if the test has been made after a lengthened period of deficient rainfall, that the required supply is maintainable. There are, however, certain strata in which this contention does not hold good, as, for instance, in rocks which are to a greater or less extent cavernous where water has been stored in larger quantities than can be removed by three weeks' pumping; but even in such cases the storage, unless of very great extent, is likely to be depleted to a sufficient extent to betray the abstraction by the non-return, or the very slow return, of the water in the well to its original or rest level.

The flow in rivers and streams varies not only from year to year but from month to month, and from day to day, the fluctuations, especially in mountainous districts, being very great, but it is only in the event of the minimum discharge being insufficient for the required supply that these changes in volume are of importance to the engineer.

Investigation of Head.—If the source is found to yield a sufficient regular supply, it becomes necessary to find out whether the water can be taken from it at an elevation which will enable it to be transmitted by gravitation to the places where it is to be used, or whether it will be necessary to resort to pumping in order to obtain the necessary head to overcome the friction in the pipes, and to maintain the necessary pressure.

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When the supply is drawn from a deep well, pumping becomes a necessity in order to raise the water to the surface, or, if need be, higher. A regular supply, not requiring to be pumped, may, however, in some cases be obtained from a river, the extreme dry weather flow of which is in excess of requirements.

Engines and Pumps.—If pumping be necessary, the size and design of the engines and pumps must next be considered, with reference to economy in working, as influenced by the quantity of water to be pumped, and the heights to which it has to be raised.

Variable Supply, and Impounding Reservoir.—If the supply be variable, it is then necessary to decide whether it will be possible, by storing the excess of the water which runs off the ground during the periods of greatest rainfall and least evaporation, to meet the deficiencies of the drier months, and so to make the supply regular. The main factors in the decision are the proportioning of the area to the storage, and the cost of storing, which will depend largely on the geological formation of the area and the configuration of the surface.

A dam placed across a stream running through a narrow gorge in a valley, the bed of which is impervious, will generally afford the cheapest reservoir for storing millions of gallons of water, especially if the gradient of the stream be but slight and the valley widens out immediately above the gorge, a lake of considerable area and depth being formed by the construction of a short dam. The cost of constructing such a dam depends largely on the selection of the site, the nature of the adjacent strata, and the height to which the wall or embankment has to be carried to impound the requisite amount of water.

Compensation Water.—In calculations relating to the available quantity of water obtainable from any area for purposes of supply or power, it is to be borne in mind that it is rarely permissible to completely stop the flow of any stream, as mills may have been constructed below the site of the reservoir, or there may be fishing rights; but under any circumstances riparian proprietors have a right to receive the undiminished flow of the stream and are under obligations to pass it on undiminished to their neighbours below them; therefore water cannot be abstracted or impounded except under compulsory powers obtained by Acts of Parliament or by agreements, which in either case almost invariably necessitates the provision of

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compensation in some form or another. In the north of England, where there are many mills, compensation is generally afforded in water, it being calculated that, as a miller suffers equally from too much or too little water, if one-third of the available rainfall be allowed to pass regularly out of the reservoir, he is in a better or at least as good a position as if he received the whole of the unregulated flow. In the south, compensation is more generally afforded in money. Where compensation is given in water the capacity of the reservoir must be correspondingly increased (*see* page 65).

Quality of Water—Mode of Purification.—At an early stage in the proceedings the quality of the water must be thoroughly investigated with a view to finding out whether it contains an undue proportion of organic impurities, of bacteria, or of mineral matter likely to be injurious to the consumer either directly, or indirectly by its action on lead pipes. The water must also be tested for other impurities either in solution or suspension, which, although not necessarily harmful to life, may yet be undesirable in water required for other than domestic purposes.

If impurities be present in objectionable quantities, the next step is to decide whether they can be removed by methods which are reliable in their action and not prohibitive as to cost.

It may be regarded as practically certain that all water obtained in a thickly populated district, except perhaps that from deep wells, will, if it be not altogether unfit for use, require some degree of purification.

Filter Beds.—The ordinary method of purification is by slow filtration through sand in specially prepared filter beds, coupled with a process of previous sedimentation in settling reservoirs, and supplemented, in some cases when the water is hard, by a softening process, which is likewise accompanied by sedimentation.

The water passes, either directly or through the medium of settling tanks, from the source to the filters, through the sand, in which it percolates at a rate regulated by the head, but which is not allowed to exceed a given amount, and is continued day and night.

Service Reservoirs.—Inasmuch, however, as the water is not required to be delivered at a regular rate, owing to the intermittent demands of the population, it becomes necessary to provide small

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reservoirs, capable of storing the water filtered during the night for use when required, with a margin in case of fire or accident.

These reservoirs must be protected from pollution and kept cool, and are therefore required by Act of Parliament to be covered when situated in or near a town.

Distribution.—The water from the service reservoir is carried through mains and distributing pipes to the place where it is required. The dimensions of the pipes must be adapted to the requirements so as to permit of the passage of the maximum amount of water wanted at any point in their lengths, without material loss of pressure, and they should be so arranged that sections may be cut off by means of valves, and repairs carried out with the least possible interference with the supply.

It is desirable that, so far as is practicable, distributing pipes should be laid in circuits so as to promote circulation and to prevent stagnation in any part of the system, and for this reason pipes laid to "dead ends" should be avoided (*see* page 156).

Separate System.—It is evident that water unfit for dietetic purposes might nevertheless be used for flushing water-closets and sewers and for washing streets. As, however, the use of unfiltered water for these purposes would necessitate the laying of two sets of mains and might lead to serious consequences if mistakes were made in connections, a duplicate system is of doubtful advantage. Such a system is in use in Paris, but is not favourably reported on by the Engineer of that city.

Selection between Sources.—The selection by the engineer of the best source out of many possible sources is usually one of difficulty, and should be governed by two primary considerations, namely, those of efficiency, first, and economy, second.

Water which is seriously polluted or is liable to serious pollution and cannot be rendered pure within the usual requirements, must be condemned. Water of a moderate degree of hardness will be preferred to that which is of greater hardness, while that which is excessively soft, especially if it contains peat or acidity, has its own danger. It is destructive to pipes and steam boilers, and may induce lead poisoning; it should therefore be treated with lime. If the supply is likely to be deficient in the future, it may be necessary to abandon it unless provision can be made for extension.

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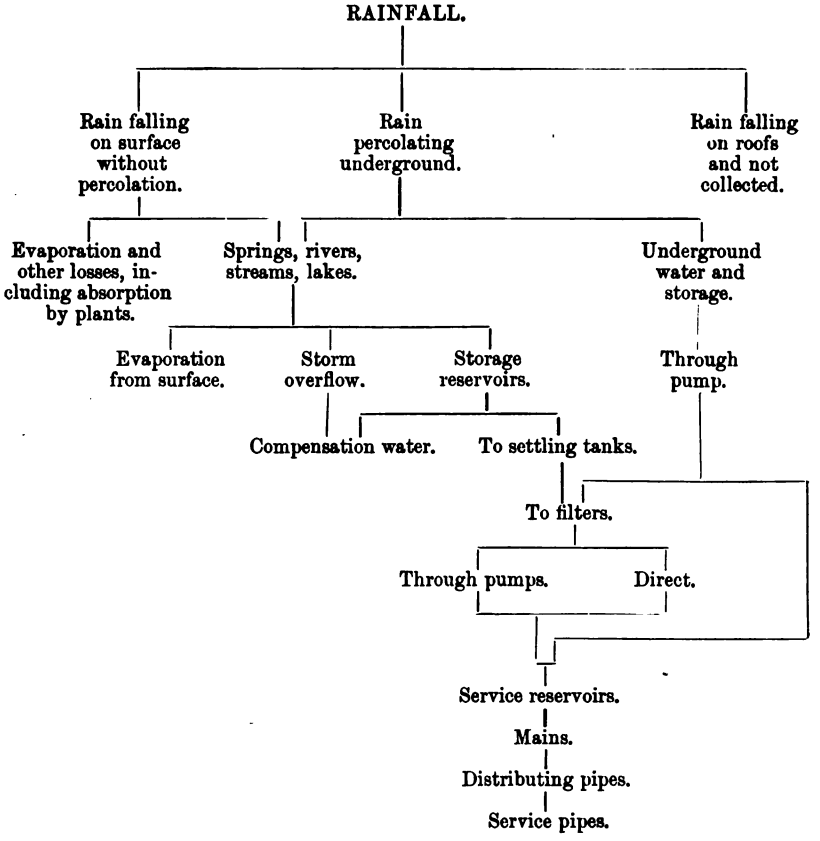
General Summary.—The gist of the foregoing considerations may be summarised in diagrammatic form somewhat as follows:—



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CHAPTER II

REQUIREMENTS AS TO QUALITY

No Water used is chemically pure.—As the preceding diagram shows, all water available for supply or any other purposes comes to the Earth from the clouds in the form of rain.

How far it can be described as pure water, immediately after condensation in the upper regions of the atmosphere, is a matter difficult of determination and of no practical importance. It is certain that, before it is collectable for use, it will be contaminated to a greater or less extent by impurities of some sort. The term 'pure' water is used in its strictest chemical sense, denoting water containing no foreign substances whatever in solution or suspension.

Even if it were possible to obtain such water, which it is not, it would not be an unmitigated advantage, as there can be but little doubt that the presence of certain impurities, in the sense of foreign matters, may, as will be seen, prove desirable rather than objectionable.

The impurities usually contained in water may be briefly classified, the system of detection formulated, and their derivation, and in what degree they are harmful, stated.

Mineral Impurities.—Impurities of mineral origin obtain access to water almost entirely during its passage over the surface and through the pores and fissures of rocks. Near the coast common salt may be blown inland from the sea, and be taken up by the rainfall, while certain salts may be dissolved by surface water in districts more or less bare of vegetation.

The larger proportion of water has, however, percolated underground to an extent varying from a few inches to many feet, and will therefore contain in solution some of the soluble salts found in the rocks (to use the geological expression) met with in its passage.

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The most frequently occurring salts are the compounds of the alkalies and of the alkaline earths, and in particular the salts of sodium (as common salt), potassium, calcium (sulphate and carbonate of lime), and magnesium.

Whence derived.—The quantity and proportion of these salts present will obviously depend upon the course of the water. In certain districts, for instance, where the water flows over clays containing large quantities of gypsum (sulphate of lime), it carries some of that substance in solution. Water from the same districts may also contain salts of magnesia.

Water passing through chalk or limestone beds will contain carbonate of lime. This substance is not directly soluble in water, but is rendered so by the presence in the water of acid substances derived from the air or from the decomposition of organic matters. Salt may be obtained from salt beds, or, near the coast, from the air.

Hardness.—The presence of certain of these salts in solution in water (particularly those of calcium and magnesium) confers what is known as 'hardness.' Salts of soda and potash do not come under this head.

Effects of Hardness—Drinking Water.—The question of how far a moderate amount of hardness in potable water is desirable, or the reverse, has been much discussed, but without any very definite result, and an examination of the inhabitants of our own country will demonstrate that there is no very material difference in the healthiness of populations drinking soft water as compared with those supplied with hard water, provided that extremes are avoided in both instances. It may, however, be accepted that, when it can be obtained naturally, water having about 5 degrees of hardness constitutes an excellent supply, but as in far the larger number of instances it is impossible to ensure any particular qualities in the water, that which is procurable must, with certain broad limitations, be taken. The water must not contain organic impurities to a dangerous extent, and the hardness should not exceed 25 degrees.

Although a public water supply is generally considered primarily from a dietetic standpoint, it does not appear that, within limits, the use of hard water as a beverage is to be preferred to soft, or *vice versa*, except as a matter of sentiment or personal preference; but there can be no question that extremely soft water, especially such as contains peaty matter and is acid, tends to destroy the lead in cisterns and service pipes, and may and frequently does produce lead-

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poisoning; and soft water of high organic purity is, if polluted, more dangerous than water of less initial purity, as it contains fewer bacteria tending to destroy dangerous organisms or to render them inert.

Although the saving to be effected in the consumption of soap by the use of soft water, on the erroneous supposition that the whole of the water must be softened, has been much exaggerated, there is a general, though not universal, preference for soft water for cleansing purposes.

Trade Purposes.—By far the larger proportion of the water supply of any population is utilised for other than dietetic purposes, as for flushing closets and sewers, for washing and watering streets, where its quality is of no moment, and for trade purposes, when its constitution may be, and frequently is, of the first importance, as, for instance, when it is used in steam boilers. Very hard water, especially where the hardness is permanent, produces a scale which interferes with the conductivity of the plates, thus causing loss in fuel; it renders the plates liable to be burnt, and the scale has to be removed at intervals, with a resulting loss of time and labour. On the other hand, very soft or acid water produces rapid pitting in the boiler plates, which is even more destructive than a thick scale; water which is either too hard or too soft is therefore to be avoided.

There are numerous other instances of the use of water for trade purposes in which the requirements are most diverse, but it is sufficient to say that as each must be considered by itself, no general remarks would be applicable.

Lead and Copper Salts.—Salts of lead and copper may be dissolved directly in formations in which these metals occur. Water containing even small quantities of such salts must be regarded as dangerous.

Suspended Mineral Impurities.—In addition to impurities in solution, river water generally contains mineral impurities in suspension, in the form of mud, silt, etc.

The quantity of this matter present will depend, naturally, upon the nature of the country through which the course of the river runs, and upon the velocity of the flow.

It is either derived from the washing away of the river bed and banks, or is washed into the river (or some stream running into it) from the surface by the rain.

It is evident that a river with a tortuous course running through

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soft alluvial soil will be likely to derive much mud and silt from the washing away of its banks.

Rivers running through cultivated country will contain much matter washed in by surface water, and so on.

How far such matter will be carried by the stream will depend principally upon the velocity. Rivers which at points in their course form lakes, and enter these lakes in a very muddy condition, may leave them perfectly clear and bright.

The velocity through the lake, in consequence of the increased area of section, is reduced almost to zero, and the suspended matter is deposited. The same process occurs in impounding reservoirs. Hence the quantity of suspended matter is a factor which will affect the level at which the water can be drawn off from such a reservoir, and must be taken into consideration.

This quantity is, however, liable to be over-estimated by inexperienced persons. The quantity of suspended matters, even in an apparently dirty river, would, if it could be all brought to the surface at any given time, so as to spread uniformly over it, be found to have a very small average thickness indeed,—*so* small as to be perhaps hardly measurable. It can, in general, be got rid of by mere deposition.

Gaseous Impurities.—The rain in its passage through the atmosphere dissolves some of the gases from it, the ordinary gases of the atmosphere being all more or less soluble.

Of these, the dissolved oxygen is very important, being necessary for the life of the various creatures which live in water, and which may exert a powerful influence on questions of water supply. The oxygen dissolved in this way—and also after the water has fallen, by contact with the air, during strong winds, etc.—also exerts a strong purifying influence, either directly or through the medium of the living organisms in the water.

A river whose water is very impure at a certain point, if allowed to flow without receiving any further pollution, will speedily lose many of its objectionable qualities by oxidation.

The dissolved carbonic acid gas (CO_2) derived partly from the air and partly from the soil, also has an important effect, inasmuch as it enables the water to dissolve carbonate of lime from limestone or chalk beds, which consist almost entirely of this substance. The carbonates of lime and magnesia would otherwise be insoluble, but, in consequence of the presence of carbonic acid gas, they become con-

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verted into soluble bicarbonates, and are often present in solution in water in greater quantity than any other mineral substance.

Temporary and Permanent Hardness.—If the water containing these gases in solution be heated, the greater part of the dissolved gases will be given off before the boiling point is reached. The bicarbonates become decomposed by the heat, and the carbonic acid gas in solution is driven off in this way as well as the oxygen. The water is no longer able to hold the carbonates of lime and magnesia in solution, and they are deposited, furring the inside of the vessel in which the water is heated. Other salts, such as sulphate of lime, which are soluble in water, independently of the presence of carbonic acid, will still remain in solution.

The temporary hardness may also be removed by adding a solution of oxide of lime (lime water) to the hard water. This substance extracts the carbonic acid gas from the bicarbonates, and so forms carbonate of lime, which is deposited together with that already in solution. (Clark's process.)

Hence the "hardness" of water is divided into (1) temporary hardness—due to the presence of salts not directly soluble in water, and deposited on boiling or by chemical agencies; and (2) permanent hardness, due to salts directly soluble in water, and not removable by mere boiling.

Other Gases.—Ammonia and other gases may also be derived from the air in small quantities, especially near towns, or they may be generated in the water itself by the decomposition of organic matter.

Vegetable Impurities.—Vegetable impurities in water are mainly detrimental by supplying food for bacteria, etc., which will be referred to later, by leading to acidity, which may cause the water to attack lead pipes, boilers, etc., as already mentioned, or by causing diarrhœa in persons drinking the water.

Water highly charged with vegetable impurities comes mainly from flat, swampy districts, or where peat is abundant. It has rarely, if ever, penetrated much into the earth, and is soft in character. It may also be much discoloured by the decaying peaty matter, sometimes having almost the appearance of porter.

Animal Impurities.—Under the head of animal impurities we may include such pollution as enters the water after it reaches the ground, and has its origin from human or other animal sources, either directly or indirectly. Of these, by far the most important is human sewage.

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To what extent diseases are water-borne is uncertain, but two, typhoid and cholera, are generally accepted as being transmitted, directly or indirectly, through the medium of water, and it is generally believed that transmission can only take place from one body to another, therefore the presence of human fæces or urine from an infected person are sources of danger which should be guarded against, and for this reason the dejecta of persons lying in hospitals and suffering from diseases which may be spread through the medium of water, are not turned directly into the sewers, but are effectually sterilised before being disposed of.

The importance of preventing the undue pollution of sources of water supply has been recognised, and measures having this object in view are embodied in several Acts of Parliament, of which the most important should be the several public Acts for the prevention of pollution of rivers, and the private Acts which have been introduced with the same object. It is unfortunate, however, to find that for practical purposes the earlier public Acts for the prevention of pollution of rivers are almost inoperative, while even those of later date do not give satisfaction to the sanitarian.

Manufacturing and Agricultural Waste.—The subject of the disposal of manufacturing waste is also important, as such waste is liable to introduce into rivers and water-courses substances either directly poisonous or otherwise injurious. It is evident that to discharge the waste-products of dye-works, for instance, where many poisonous chemicals may be employed, without suitable and sufficient treatment, into a river the water of which is to be used for drinking, would be injurious to the public health unless the dilution were so extensive as practically to obliterate the taint. Hence a preliminary treatment with the object of partially purifying such wastes is provided for in the Acts referred to, though perhaps inefficiently.

Another more or less serious source of pollution is the washing into streams, from cultivated or pasture lands, of either natural or artificial manure.

This is less serious than human sewage, as being less concentrated, and less likely to contain germs directly productive of human diseases.

Bacteria.—Bacteria, regarded as impurities in water, are here given a separate heading on account of their importance, it having been proved that many diseases to which human beings and other animals are subject are due to their action; and, farther, that each

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specific disease which has been shown to be caused by bacterial agency is always caused by the same species of bacillus.

Non-pathogenic Germs.—Besides the pathogenic bacteria to which certain diseases have been traced, by far the larger number of micro-organisms are not only non-injurious and non-productive of disease, but are even indispensable to healthy existence.

They exist plentifully in the air, in the soil, in the bodies of human beings and animals, in nearly all natural water, and, in fact, are widely distributed everywhere.

They may be called non-pathogenic germs, in contradistinction to the pathogenic or disease-generating organisms already referred to.

As in the visible world the struggle for existence is constant and severe, so it is in the world of minute organisms revealed by the microscope. The struggle between competitive forms is, as usual, largely a matter of food supply, although, naturally, other conditions of environment exert important influences in determining the preponderance or otherwise of specific forms. In chemically pure water the total absence of any extraneous matter that could constitute a food supply would preclude any organisms, subsequently introduced, from multiplying; but such water is only met with under exceptional experimental conditions. On the other hand, ordinary water invariably contains foreign matters suited to the development of some forms of bacteria. Hence, if what are termed pathogenic bacteria—*i.e.* forms with which certain specific diseases appear inseparably connected—obtain access to water comparatively free from other organisms, they will, under favourable conditions of temperature, etc., and *in the absence of strenuous competition*, increase until they reach the food limit. If, however, the water to which they have obtained access has already been contaminated, and contains numerous bacteria of other species, the conditions are less favourable, and the numerical superiority of the earlier arrivals will probably prove inimical to the development of the later organisms. Under these circumstances, spore-bearing bacteria may be rendered inert,[1] while those that do not produce spores may perish altogether.

Deep Well Pollution.—River water contains enormous quantities of bacteria. Deep well water, which has filtered through great thicknesses of underground strata will, on the contrary, contain comparatively few. Hence, should sewage by any means find its way

[1] "Inert" must be carefully discriminated from *destroyed*, as the activities of such organisms may be revived under altered (*i.e.* favourable) conditions.

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into the water of such a well, the pathogenic germs contained therein may find themselves placed under circumstances favourable to their development, and without the presence of any considerable number of harmless and competing forms to limit their development.

This fact is all the more important, inasmuch as water from these sources is often used without filtration in consequence of its great natural purity.

Volume to be considered.—It should be apparent from what has been said, that the question of the volume of the water into which any specific pollution is introduced is also a matter of great importance, not only directly on account of the greater dilution if the volume of water be greater, but also indirectly by the influence of the degree of dilution upon the development of the germs.

Thus a specific pollution introduced into a large river, and producing no serious consequences, might result in very serious epidemics if introduced in a small stream or well.

This question is doubtless also influenced by the fact already mentioned, that a river of large volume will contain many bacteria and much dissolved oxygen, gathered in its previous course.

Another point worth mentioning in this connection is that in upland or sparsely populated districts—just where large rivers are least likely to be found—less care will generally be taken in the disposal and treatment of the sewage, in consequence of the sparseness of the population, than in more thickly populated districts. Thus the pollution from sewage in such districts assumes an added importance, first because it is in a less innocuous state; and secondly, because it is introduced into a smaller volume of water, and under conditions more favourable to the development of the pathogenic germs.

Examination of Water.—The engineer should not fail, then, to obtain the opinion of a competent water analyst on the quality of the water he proposes to supply before deciding on any particular source. A thorough examination of the water must be made, including physical examination, chemical analysis, microscopical and bacteriological examination, and a study of the history of the water.

Collection of Samples.—It is of the utmost importance to ensure that the vessel in which the sample is collected should be clean. It should be well washed — including the stopper — with water from the same source as the intended sample.

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The most suitable vessel is a bottle containing about half a gallon, though smaller quantities may sometimes suffice. Care should be taken, in selecting the sample, that it is an average one. It will be evident, for instance, that a very great deal of difference will probably exist between two samples of water taken from the same river at spots quite close together, but of which one was just above the outlet of a sewer and the other just below it.

A clean, well-fitting stopper should be employed, and it is preferable not to seal the bottle round the stopper, but to tie the latter down by a strip of clean cloth or paper, which may be sealed below the mouth of the bottle if desired.

When a source has been provisionally selected, several analyses of the water, taken at different points and at different times, should be obtained, inasmuch as the quality of the water varies, often most curiously, not only from point to point, but also from time to time. Thus if the river is in flood, the quality may be very different from that of the dry weather flow. If possible, the analyst's advice should be secured as to the exact spots where those samples are to be collected, and he should be made fully acquainted with all that can be ascertained with regard to the history of the water. In short, he must be supplied with all possible data which can assist him in giving a reliable opinion as to the quality of the water.

Physical Examination.—In cases of extreme pollution, a mere physical examination will show that the water in its natural state is quite unfit for use. The colour should be noted. The dark colour imparted by vegetable pollution has already been mentioned. In certain rivers it is impossible to avoid noticing the odour of the water, and it is easy to arrive at a conclusion regarding the quality. Lastly, if the water is very dirty, some may be allowed to stand, and the quantity and nature of suspended matter which settles from it will then become apparent.

Minute fragments of paper, cloth, thread, etc., are also possible evidences of sewage pollution, and should be noted.

Specific Gravity.—The specific gravity of the water, as compared with that of distilled water at the same temperature, affords an indication of the total amount of foreign matter—particularly of mineral salts and carbonic acid—in solution.

The increase in density caused by mineral salts depends upon the strength of the solution, and also to some extent upon the particular salt. Thus one gramme of carbonate of potash dissolved in a litre

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of water will cause an increase in the density of the water amounting to about one and a half times as great as that caused by the same quantity of nitrate of potash. There is a very slight increase in volume in each case, so that in neither case will a litre of the solution weigh exactly 1001 grammes.

The specific gravity of ordinary drinking water should range from about 1000 to 1000·5, distilled water being taken as 1000.

Prof. Wanklyn, in his book on *Water Analysis*, gives the following approximation as applying to common salt and carbonate of lime :—

"The number of centigrammes by which one litre of the water exceeds one kilogramme expresses the number of grains of mineral residue and carbonic acid in one gallon of the water."

Test for Hardness.—The hardness of water, as already explained, is caused principally by the salts of calcium and magnesium held in solution.

These act upon a solution of soap, so as to decompose the soap and form insoluble compounds of lime and magnesium, whereby the immediate formation of a lather is prevented.

The test for hardness, then, consists in making up a 'standard solution' of soap of such strength that one cubic centimetre will be just sufficient to precipitate one milligramme of carbonate of lime. This soap solution is gradually added to a known quantity of water to be tested, the whole being well shaken after each addition.

The quantity of the solution just required to produce a *permanent* lather is noted, and by making a deduction for the quantity required to produce a lather in the same amount of *pure* water,[1] the hardness expressed as milligrammes of carbonate of lime in the quantity of water tested can be easily determined.

By selecting the volume to be experimented upon, so that it shall bear to one gallon the same relation as one milligramme does to one grain, the result will give the hardness expressed in grains of carbonate of lime per gallon.

Degree of Hardness.—The hardness due to one grain of calcium carbonate per gallon of water is said to be *one degree* of hardness.

As part of the hardness may be due to magnesium salts, the soap test does not directly tell the number of grains of magnesium carbonate per gallon, but reduces this to the equivalent quantity of

[1] On "Clark's scale" this deduction is not made. Prof. Wanklyn expresses the hardness simply by the number of soap measures required to produce a permanent lather in one gallon of water.

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calcium carbonate (equivalent, that is to say, so far as decomposition of the soap is concerned). One grain of magnesium carbonate is, in this respect, equivalent to about $1\frac{3}{4}$ grains of calcium carbonate.

Limits of Hardness.—The hardness of water for steam generation and ordinary purposes should not exceed 5 degrees, equivalent to five grains of calcium carbonate per gallon. Water is probably more suitable for general purposes when its hardness is not below 3 degrees.

Hence the limits of hardness should be from 3 to 5 degrees for the best results. It is often greater than this, but about 15 to 25 degrees should be the limit.

Beyond about 10 degrees of hardness it may be desirable to get rid of the temporary hardness by 'Clark's process,' mentioned on p. 11, or by some other similar process.

The hardness does not afford a reliable indication of the total mineral matter in solution, inasmuch as many salts of soda and potash, such as the carbonates, tend to soften the water rather than to harden it. These are determined by evaporating to dryness a known volume of the water, and weighing the residue.

Magnesium Salts.—The soap test does not act immediately with magnesium salts, as it does with lime, but requires time. Hence great care must be taken to see that the lather is really permanent, and does not disappear on further shaking.

Organic Matter.—The amount and nature of the organic matter contained in water furnishes an important indication as to its fitness for use.

Such matter may consist of living creatures (animalculæ), of sewage or the remains of sewage, of vegetable pollution, or of other decaying animal or vegetable matter washed in from the surface, or from cemeteries, cesspools, etc.

Free and Albuminoid Ammonia.—The oxidation of such matter has been already referred to under the head of dissolved oxygen (page 10).

Thus, taking sewage as an example, the urine in it very readily becomes decomposed, yielding at once a certain amount of 'free ammonia,' either as ammonia gas, or as carbonate of ammonia, which will remain in solution.

In addition to these, a certain amount of the organic matter in the urine will remain undecomposed for some time. This, together with other organic matter in the sewage, will ultimately become

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oxidized by natural processes; or it may be at once decomposed by the addition of certain chemicals, so as to yield further ammonia.

The ammonia yielded by previously undecomposed organic matter serves as a measure of the latter, and is known for test purposes as 'albuminoid ammonia.' It must be understood, however, that the number of grains per gallon of albuminoid ammonia is less than the number of grains of organic matter, the proportion being also variable, inasmuch as the ammonia contains nitrogen, and some kinds of organic matter are more nitrogenous than others.

Estimation.—The organic matter in the water is estimated by determining the quantities of free and albuminoid ammonia present.

Under the older methods of examination the organic matter was determined by evaporating a quantity of the water to dryness, and igniting the residue after weighing. The organic matter was literally *burnt* out, and the loss in weight was taken as an indication of its quantity. Inasmuch, however, as there are other volatile matters—including water of crystallization—in the residue, the test was not satisfactory.

At the present day the quantity of organic matter is sometimes determined by what is known as the 'moist combustion process,' whereby the quantity of an oxidizing agent (permanganate of potash) required to oxidize it, is measured.

The more common test, for ammonia, is performed by adding to the water a strongly alkaline solution of iodide of potash saturated with periodide of mercury, known as the 'Nessler reagent,' from the name of the inventor of the test.

Any ammonia present in the water will react with this solution, causing a brown coloration, the depth of which depends upon the quantity of ammonia, and hence serves as a measure of the latter. The actual amount present in the sample under investigation is determined by comparing the depth of coloration produced by that arising from known quantities of ammonia separately tested.

The water to be tested is first submitted to distillation. All the free ammonia present will distil over with the first third or thereabouts of the water taken, and this is tested separately.

The result gives the quantity of free ammonia, or recently decomposed organic matter.

The remainder of the water is then treated with a solution of potash and permanganate of potash, which brings about the decom-

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position of the remaining organic or albuminoid substances, liberating more ammonia.

This, on further distillation, comes over with the distillate, and the latter is tested, as before, in portions, until no more ammonia is found in it.

The result gives the quantity of 'albuminoid ammonia,' whence the undecomposed organic matter is calculated.

Conclusions to be drawn.—As will have been gathered, 'free' ammonia is formed by the immediate decomposition of contaminating matter in the water. Hence a quantity of it, together with the presence of albuminoid ammonia, points to recent contamination with readily decomposable substances—particularly urine. The contamination was most probably very recent, if the free ammonia is very high, inasmuch as the ammonia soon becomes oxidized or decomposed, yielding nitrites and nitrates or free nitrogen. Free ammonia continues, however, to be yielded by the decomposition of the other organic matters.

'Albuminoid' ammonia is derived from substances less easily decomposable—and which may therefore remain in the water for some time—and from such living creatures as find their food in these substances.

These will be found in the water after the greater part of the free ammonia derived from the rapid decomposition of urine has disappeared, and hence the presence of albuminoid ammonia in quantity, and little free ammonia, points to contamination either of an earlier date, or of recent date, but containing little urine.

This may, for instance, be evidence of vegetable contamination or the like, which is less likely to immediately yield free ammonia than is sewage containing urine.

Such vegetable contamination is injurious to health, though, as already pointed out, less so than sewage contamination.

Limits to be allowed.—The quantity of free ammonia should not exceed about ·08 part per million, as above this figure it almost certainly indicates, according to Prof. Wanklyn, recent contamination with urine.

In water which has been treated by filtration, the amount found is almost invariably lower than this.

The quantity of albuminoid ammonia should be less than ·1 part per million.

Efficient filtration will always reduce it below this figure in any

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ordinary water. Hence, if it is to be filtered, it is not absolutely necessary that a water should be condemned because it contains more than this in its raw state.

Unfiltered Thames Water.—Prof. Wanklyn, in his book on *Water Analysis*, mentions a sample of unfiltered Thames water taken from near Hampton Court as containing ·04 part per million of free ammonia, and ·28 part per million of albuminoid ammonia. At London Bridge at two hours' flood the figures were 1·76 for free ammonia, and ·35 for albuminoid.

The first named sample may be taken as giving the quality of the greater part of the London water supply before filtration. After filtration by the New River Company, for instance, the figures were on the average ·01 and ·07.

Chlorine.—The quantity of chlorides in the water is usually determined also, the reason being that as sewage—and particularly urine—contains a considerable quantity of sodium chloride in solution, the presence of chlorine may point to pollution from this source. It is to be remembered, however, that water may flow over beds of salt or may obtain salt from the sea. Hence the presence of chlorides in the water proves little of itself, but it is *suspicious*, and calls for a further study of the history of the water.

It is confirmatory of sewage contamination when associated with free or albuminoid ammonia.

Conversely, the *absence* of chlorides indicates almost certain freedom from *sewage* contamination — but not from other kinds of pollution.

The test for chlorine is performed by finding out how much nitrate of silver is required to combine with it.

The chloride of sodium is not removable by filtration, and the quantity of chlorine in drinking water may be as high as about two grains per gallon without inconvenience.

Nitrates and Nitrites.—The organic matter contained in the water becomes, as already stated, gradually oxidized, much of it being converted into nitrites, and ultimately into nitrates. Hence the presence of nitrates may be evidence of organic pollution.

The same considerations arise in this connection as in the case of chlorine, except that the presence of nitrates does not necessarily mean *sewage* pollution.

Salt occurs in sewage, but nitrates may be formed from any organic matter by oxidation, or may be derived by mere solution

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from the soil, and may be found without any possibility of organic pollution.

Conversely, absence of nitrates is not conclusive evidence of the absence of organic pollution, as even when formed they may have been subsequently absorbed by plants.

There are springs the waters of which are clear, bright, sparkling, and yet loaded with nitrates, and which in their course have run near or through cemeteries.

This accounts for the presence of nitrates. Such water may not produce any injurious effects if used for drinking, but it is evident that if so used, and if at any time the water flowing through the cemetery should find its way into the springs by an easier route than that it now follows (one, that is to say, accompanied by less efficient natural purification) very serious results might follow.

Hence the presence of nitrates, as of chlorine, should lead to the further study of the history of the water. The test for nitrates is performed by reducing them back again to ammonia, which is determined as before.

Poisonous Metals.—The presence of lead and copper in water has already been referred to. These substances are probably little, if at all, lessened in quantity by filtration; and hence their presence in any quantity (0·1 grain per gallon) must condemn the water, unless they can be removed by chemical means.

Zinc may also be looked for, but is probably more likely to get into the water—especially soft water—from pipes, etc., after collection than before, except in special cases.

Organic Carbon.—The quantity of organic carbon—as distinguished from the carbon in combination as carbonates or carbonic acid—is also sometimes determined.

If it be plentiful in comparison with the ammonia and chlorine, it will probably point to vegetable pollution; if not, the pollution is probably from an animal source, inasmuch as organic matter of vegetable origin contains relatively more carbon and less nitrogen than animal substances, whence a given amount of albuminoid matter will mean more organic matter if the latter be vegetable than if it be animal matter.

This determination is of less value, however, than those already referred to, and is not always made.

Microscopical and Bacteriological Examination.—A simple microscopical examination of water will often show the presence in

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it of many living organisms. Of these some may be directly harmful. This class includes not only the pathogenic bacteria already referred to, but other organisms also. Thus immature forms of 'Entozoa' are frequently found in water.

Some of the forms present may not be directly harmful, but they all indicate more or less organic pollution.

As far as the purely bacteriological examination goes, a process of cultivating such germs in, or on, sterilized nutrient matter is the method employed. It is difficult, however, to distinguish between the different kinds of germs which the water contains. The conditions of development are artificial and unnatural; and, as we have seen, the mere fact of there being bacteria of some sort in the water need not condemn it, as many of these are protective rather than injurious.

Hence the bacteriological examination really tells but little of value. Even if a water containing much organic matter *should* appear, on examination of a sample, to contain no pathogenic germs, the use of that water in an unfiltered condition, for instance, would be distinctly inadvisable, inasmuch as the absence of the germs in the particular sample of water collected does not conclusively prove their absence in the whole of the water. Nor does it prove that they will continue to be absent; and the organic pollution, of which their presence might have been an indication, must still be regarded as a possible danger.

History of Water.—It is hoped that the preceding remarks will have sufficiently indicated the principal impurities to be looked for in water, and how far they are objectionable.

The presence of almost any given impurity may, as has been shown, be due to any one of a number of causes, and in many cases it is the question of the origin of a given impurity that matters, rather than the actual presence of that impurity.

In discussing these subjects, it has been necessary to refer to the necessity that exists for knowing something of the history of the water.

In order to put this matter in a clearer light, however, we will now consider the subject specially from this point of view, looking at the water according to the source from which it has been obtained.

Stored Rain Water.—Rain water collected from roofs, etc., is, in general, both inefficient for water-supply purposes, and, to some extent, dangerous. Its quantity is variable, and at best, in most

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districts, not equal to the requirements. If convenient means for storage can be obtained, it may, however, be used for washing and like purposes where the ordinary supply is hard or insufficient.

The first water collected after a long drought is foul. Messrs. Parkes and Kenwood, in their work on *Hygiene and Public Health*, state that it may contain 200,000 germs per litre, found chiefly in the form of spores. For this reason rain water separators are sometimes employed, which cause the water from the first showers to run to waste.

Apart from these, and from the presence of such gaseous impurities as the water will derive from the air, such water will be fairly pure, and has but little history.

Upland Surface Waters.—Next to rain water directly collected, 'upland surface water' has the shortest history. It may be defined as the water falling on the soil in mountainous or other *uncultivated*, and therefore unmanured and probably uninhabited districts, and flowing by gravitation into the natural drainage channels of the district, there forming streams, mountain lakes, etc. whence it is drawn.

The water being collected and drawn off in uncultivated—and therefore almost, if not quite, uninhabited—districts, is not subject to much sewage pollution.

Being surface water, it has not penetrated much into the soil, and hence contains little mineral matter in solution.

The principal impurities to which it is subject are of vegetable origin, and it will contain comparatively few bacteria.

The principal disadvantages are that such water is likely to develop the property of attacking, corroding and furring up any metallic pipes or cisterns into which it may be led, and a specific pollution, *i.e.* typhoid germs, etc., is more likely to lead to a serious epidemic, owing to the multiplication of the pathogenic germs being favoured by the absence of other germs; while, finally, unless the supply is drawn from a lake, an impounding reservoir will almost always be required, as the flow depends so directly on the immediate rainfall, and hence is so very variable.

It appears that vegetable impurities can be very largely removed or rendered innocuous by sedimentation and by sand filtration (to be afterwards described), and the water may be hardened, if desired, by the use of a certain amount of lime in the filters.

Many large towns and cities in the United Kingdom are supplied

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with such water, and on the whole, unless drawn from very flat swampy districts, upland surface waters may be regarded as forming a desirable source of supply.

The nature of the mineral impurities in them will depend on the character of the surfaces from which they are collected, as will also the quantity to some extent.

If the surface on which the water falls consists of impervious rocks, say granite or the like, at a steep slope, it will rapidly find its way into the nearest stream in an almost pure condition, containing probably much less than 1 degree of hardness, and little organic matter. Some small amount of chlorine may be present.

If the surface consist of softer or more soluble rock, as limestone, the hardness may be greater, but the water will still be fairly pure.

Water from flat clay or alluvial surfaces will generally be less pure, but such flat surfaces are seldom uncultivated. It will contain possibly from 2 degrees up to even 40 degrees of hardness. Water from such clay surfaces is also somewhat highly charged with suspended matter, and may contain more organic matter, due to vegetable growth.

Finally, there is the water from swampy districts or peat bogs. This will contain hardly any hardness, but much organic matter.

Surface Waters from Cultivated Ground.—The water falling on the surface of cultivated—and therefore inhabited—ground, and collected therefrom, would have a history almost as simple as that falling on upland surfaces but for the fact that it runs into streams or rivers at points which are, in general, at some distances from their sources. The streams have therefore acquired much extraneous matter, and this will be added to the impurities present in the fresh surface water.

Cultivated land is always flat or of moderate slope, and much of it is ploughed. Hence the surface waters will run off slowly, and will come into contact in their flow with much finely divided soil. They may thus contain 10 degrees (or more) of hardness, and a proportion of the substances, if any, used as manure on the land.

These waters will, when the land is flooded, contain some organic matter both of animal and vegetable origin swept into the streams, hence water collected from cultivated, and therefore inhabited, land must be looked on with greater suspicion, examined with

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greater care, and protected more thoroughly than that which comes from uncultivated areas or deep wells; nevertheless, where proper safeguards are employed, distinct pollution is prevented, sedimentation is provided, and filtration is carried on on the most approved system, water obtained from even highly cultivated land may be used with perfect immunity from danger, owing to the fact that all water which falls on cultivated lands, which must be permeable, penetrates to a greater or less extent, and, except at times of flood following a lengthened period of dry weather, washes the impurities into the soil, which forms a natural mechanical and biological filter, and will purify it before it reaches the river.

Water will, of course, penetrate below the surface wherever the latter is permeable, whether the ground be cultivated or not, and will then simply follow the line of least resistance, sinking until it meets an impermeable stratum, and then flowing by gravitation towards the lower ground.

Occasionally it finds its way into a natural basin or hollow in the underlying impervious stratum. It then forms an underground reservoir analogous with a pond or lake above ground in that the water cannot escape below a certain level, but different in that the upper surface is not horizontal and the water does not generally enter or leave it by any well-defined channel, but simply oozes through every available pore in the saturated ground.

Such water may travel along underground for a short or long time and at a small or great depth, ultimately reaching the surface lower down the country in the form of springs, or be partially drawn up through wells.

In its course it may when it approaches the surface have come into contact with decaying animal matter in cemeteries, or it may possibly have passed near and under privies, cesspools, or defective sewers, and hence been contaminated by leakages from such sources, but as this must almost of necessity take place near the surface, it can be traced and guarded against. If it should enter a well just after this contamination reaches it, a dangerous condition of affairs may result.

If contaminated by sewage, the water from such a well will generally be loaded with chlorides, besides containing much free and albuminoid ammonia.

If the pollution be of vegetable or other origin, but not from sewage, the chlorides will be less in quantity.

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Specimen Analyses.—Prof. Wanklyn gives specimen analyses of such water, containing in parts per million:—

Number.	Chlorine.	Free Ammonia.	Albuminoid Ammonia.
1	7	·02	·34
2	80	1·20	·08
3	80	0·0	·84
4	140	32·8	·52

No. 1, containing little chlorine or free ammonia and fairly high albuminoid ammonia, we should infer to be probably polluted with vegetable matter. No. 2, from its high ammonia and chlorine, shows possible recent contamination by urine and sewage, while No. 3 contains high chlorine, but no free ammonia. This would point to more remote contamination by urine, and high nitrites or nitrates would tend to confirm this view. The high organic ammonia, in any case, would cause the water to be looked upon with suspicion.

The proportion of completely oxidized nitrogenous matter, as nitrates, may also be as high as 8 parts per million.

Such water (especially if a fair proportion of the organic matter be completely oxidized) may be bright and sparkling when drawn, and even very palatable, but if kept bottled up for some little time it may nevertheless become turbid and foul-smelling.

Positions of Wells. Flow of Underground Water.—Now let fig. 1 represent a section, in which GH is the 'line of saturation' or level to which the water rises, O being a well.

Under ordinary circumstances the upper surface of the water would lie along the line GH, and borings would show that this had a slope in some direction, this slope depending upon the head required to force the water forward through the strata.

Now if water be withdrawn from the well while pumping is going on, the level of the water inside the well will sink to some point EF, such that the rate of flow of water into the well due to this depression will be equal to the rate at which the water is being pumped out.

As the level sinks, the water converges towards the well from the

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water-bearing region on all sides of it. It thus moves in ever narrowing circles, and therefore with an ever-increasing velocity. Hence the gradient of the lines ECG, FDH, which represent the altered position of the line of saturation, will get steeper as these lines approach the well.

Horizontal sections of the ground at different levels would show that while the pumping proceeds the water is more or less completely

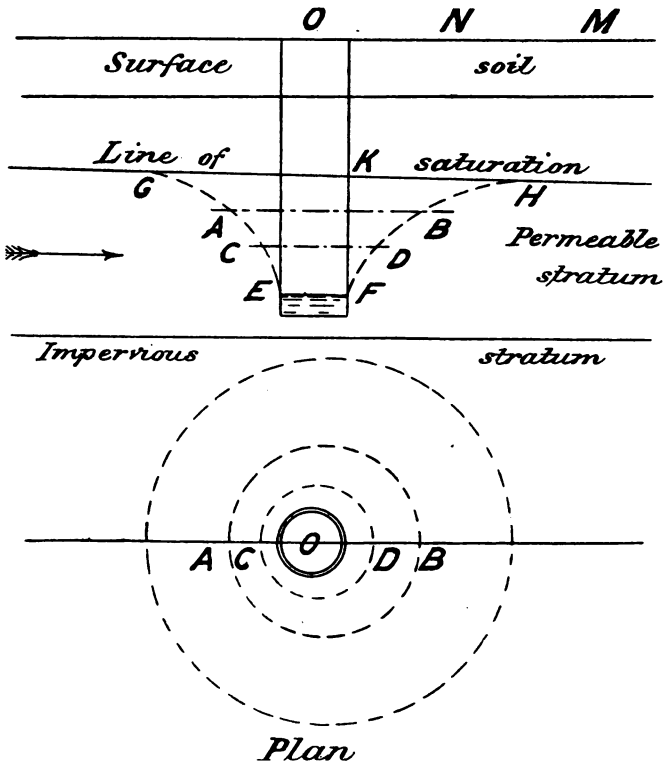

FIG. 1.

removed from a part of the ground approximately in the shape of a circle with its centre coinciding with that of the well, the diameter of this circle decreasing as the depth gets greater.

Hence the water is more or less completely withdrawn from an area roughly in the shape of an inverted cone, having its vertex at the bottom of the well.

This is known as the cone of exhaustion, and if any source of pollution exist within this area, either above or below the well (as at

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N), it will affect the water drawn from the well. Pollution below the well, but outside of this area, as at M, would not sensibly affect the water. Any pollution on the 'up-stream' side might of course affect the water whether inside the area or not.

The arrow shows the direction of flow. Theoretically the curves ECG and FDH should become 'asymptotic' or tangential to the original line of saturation, and the withdrawal of the water should cause a lowering of the level all along the line, both above and below the well, but in general, for one well, this is scarcely noticeable. The line of exhaustion also approaches somewhat nearer to the well on the up-stream side than on the down, in consequence of the normal velocity of flow.

Hence there is much to be studied in selecting the exact site for such a well. The direction of flow of the underground water must be

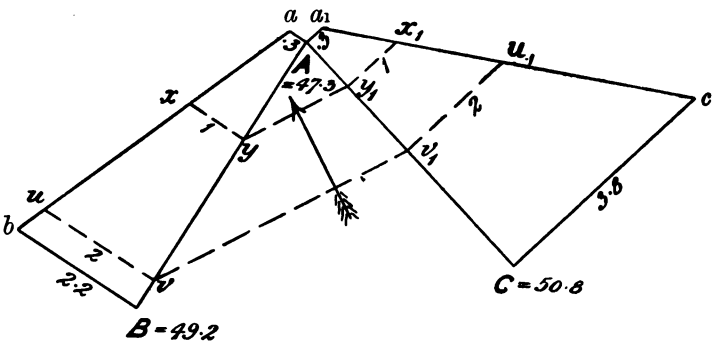

FIG. 2.

determined by borings or by examination of existing neighbouring wells. By levelling from one to another, and taking the height of the water in each, the direction of the flow and the slope of the subterranean water can be ascertained.

From three such levels, at the corners of a triangle, a simple construction will give an approximation to the direction of flow. The well must then be placed on the *up-stream* side of all sources of pollution which can be discovered.

To get the approximate direction of flow, suppose A, B, C, fig. 2, are three points on the plan whose (water) levels above any datum are found to be—

For A, say	- - -	47·3
„ B, „	- - -	49·2
„ C, „	- - -	50·8

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Take AB as 47 above datum, and plot

$$Aa = \cdot 3 \ (= 47\cdot 3 - 47)$$
$$\text{and } Bb = 2\cdot 2 \ (= 49\cdot 2 - 47),$$

both perpendicular to AB.

Join ab. Then find the points y and v in AB such that $xy = 1$ and $uv = 2$, both being perpendicular to AB.

Similarly taking AC as 47 make—

$Aa_1 = \cdot 3$, and $Cc = 3\cdot 8$ $(= 50\cdot 8 - 47)$, and find the points y_1 and v_1 whose ordinates to a_1c are respectively 1 and 2 units to scale.

Then yy_1 and vv_1 will be level lines on the surface of the water, and the general direction of flow will be perpendicular to these as shown by the arrow, but the actual direction may be curved.

Deep Wells and Deep Springs.—If water contaminated in the manner described should, instead of being withdrawn soon after, continue in its course underground, sinking to depths where it is little likely to meet with further organic pollution, a process of natural purification will go on, so far as the organic matter is concerned, often resulting in the elimination of the latter before the line of saturation is reached. The purification will be of a twofold character, viz., first, a process of mechanical filtration whereby any sedimentary matter is removed; and secondly, a process of oxidation.

Organic matter, in general, consists of carbon, nitrogen, and hydrogen in a state of partial oxidation, together with certain other chemical elements, such as phosphorus, sulphur, potash, etc.

When it decomposes in nature, the greater part of the nitrogenous matter will form ammonia, and this may combine with water and carbonic acid, formed by the oxidation of the hydrogen and carbon, so as to form carbonate of ammonia.

In the presence of metallic bases and more oxygen the process of oxidation will, however, continue. The carbonate of ammonia is split up, and metallic nitrates are formed; while it is probable that in nature the carbonic acid thus liberated from the ammonium carbonate, combines with normal and insoluble carbonates already in the earth, so converting them into soluble bicarbonates, as already stated.

The oxidation of the organic matter being completed, the bacteria which caused it will have no more food supplies and will gradually either die of starvation, and become oxidized in turn, or be filtered out of the water by the strata it passes through.

All this, then, goes on in those waters which sink in nature below the level of organic pollution. If it continue long enough

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the water will ultimately contain practically no organic matter, and its only impurities will be the mineral matters (nitrates, chlorides, etc.) obtained from the original pollution directly or by oxidation, and such other salts as it has derived from the earth, and will vary with the nature of the strata through which it has passed.

Water from chalk contains little permanent hardness. The mineral matter will be nearly all in the form of bicarbonate of lime, and this alone may contribute over 20 degress of hardness.

Water from limestone beds contains more permanent hardness—due to sulphates chiefly—than do chalk waters. If the district contain dolomite, much of the hardness will be due to magnesium salts.

Water from the lias clay and similar strata is generally rich in permanent hardness, and not to be compared in its general features, such as brightness, with the waters already named.

If the water be collected from loose sand, gravel, or ordinary sandstone formations, it is likely to vary much in the impurities it contains, these strata being deposited under varying conditions, and therefore being of variable composition. The hardness may vary in amount, but is usually mainly permanent. The 'total solids' in solution (including salts of potash, soda, etc., as well as those producing hardness) may reach 1 part in 1000.

Messrs Parkes and Kenwood, from whose book on *Hygiene and Public Health* the above classification is largely taken, also point out that the greensands contain reducing salts of iron which may reduce the oxidized nitrogen (*i.e.* the nitrates) so as to yield free ammonia.

Specimen Analyses.—Prof. Wanklyn in his *Water Analysis* gives the following analyses of water from deep wells:—

	Grains per gallon.		Parts per million.	
	Total solids.	Chlorine.	Free ammonia.	Organic ammonia.
No. 1.	26·6	2·1	·03	·02
No. 2.	27·1	2·5 (about)	·005	·005
No. 3.	54	9·7	·82	·04

No. 3 was from a deep well at Blackfriars, and furnishes an example of free ammonia in a pure deep well water.

Danger in Deep Wells.—Thus in general the water from a deep well contains hardly any substance likely to be injurious for potable purposes.

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Its principal objection is its hardness, which, if the water is to be used in boilers, etc., may in general be partially removed by Clark's method or some other processes.

It is to be borne in mind, however, that this very fact constitutes a danger, as it is liable to lead to a sense of false security in the use of the water.

The rule, like nearly all others, has its exceptions. These arise mainly in one or other of two ways:

(1) The water in its passage through the earth *may* meet with fissures or distinct channels in the rocks, through which it will flow rapidly, with comparatively little purification, towards the well.

(2) The water, after having become properly purified before entering the well, may be, by some accident or otherwise, polluted *after* entering the well.

This is perhaps the most dangerous case of all. The very purity of the water, its freedom from bacteria, and its position deep down in the well, will lead to the rapid development of the germs contained in the pollution.

It is to be borne in mind also, that the fact that the well yields pure water to-day does not prove that it will do so to-morrow.

At any time a leak, fissure, or open channel may be formed—possibly by the water itself—leading either to imperfect purification of the main body of the water, or to the entrance into the well of the drainings from some source of pollution which were previously excluded.

Rivers, Streams, and Lakes in Cultivated Lands.—Rivers in general will be a mixture of the waters from all the sources already considered, hence little more need be said about them.

We have seen that the process of purification takes place, so far as organic matter is concerned, by the successive action of numerous species of bacteria in three main stages. In the first, ammonia is formed. In the last two, this ammonia is oxidized. The latter involves two stages by the action of bacteria of two distinct groups. Those which lead to the reduction to ammonia fulfil their purpose better in the absence of light and oxygen, and hence are known as *anaerobic* organisms. Those involved in the subsequent oxidation processes are known as *aerobic* organisms.

There are also bacteria which are neither entirely aerobic nor entirely anaerobic, but which can function as either, and probably serve to carry on some intermediate processes. The intermediate

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stages and the specific bacteria engaged during each step are not, as yet, fully known.

In ordinary sewage and river water both aerobic and anaerobic germs are probably present, and if the quantity of pollution be limited, relatively to the volume of the water, the processes of purification will go on pretty much on the lines already sketched out. How far it will be effective in any case will depend upon the quantity of free oxygen in solution in the water. A slow flat stream will not dissolve so much oxygen as one containing many waterfalls, or the surface of which is subjected to much agitation.

If sewage be discharged into it at a number of points, it may happen that the oxygen in solution will be used up in the purification of the first pollution, and future purification must be accomplished by anaerobic germs entirely.

This process will be less rapid and a great deal more dangerous and disagreeable than that already described, and will lead to foul smells, much turbidity, and the possible development of harmful rather than harmless germs if such be present.

The germs of typhoid fever and cholera, for instance, are anaerobic, and their development is interfered with by a plentiful supply of sunlight and air. Many harmless germs do not appear to be interfered with by light.

Effect of Sedimentation.—Another and most important means of purification deserves mention. When there is suspended matter in the water, this in its gradual deposition carries with it many of the bacteria, etc. A very considerable proportion of the total bacteria may be removed in this way. A similar result obtains in virtue of the deposition of carbonate of lime in the softening process already described.

The point, then, at which a sample of river water is collected for analysis should be near the point at which it is proposed to withdraw the water for the proposed supply.

It may be mentioned, in conclusion, that the question of how far a river must run before it gets rid of any pollution by natural processes is a much debated point. It depends, in fact, on many conditions, hence no general rule can be given, though in certain well-known cases a flow of 15 miles has been found sufficient to remove a pollution which is small in proportion to the volume of water.

By filtration we can, however, carry the work commenced by nature to a point nearer completion, hence absolute purity in the river is unnecessary as well as unattainable.

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We have seen that the impurities in water may be varied in character and amount; that the same impurity may have a harmless or a harmful origin, hence, if it be harmless in itself, it may still be necessary to trace the history of the water to discover the origin of the impurity; that a pollution which is in a harmless form to-day may be in a harmful form to-morrow, in consequence of its traversing the distance between its origin and the water by a different route; that the harmful impurities of most common occurrence are poisonous metals, organic pollution, and hardness beyond a certain degree; and that the limiting amounts of these impurities which should be allowed in drinking water are:—

For poisonous metals, *nil*;
For organic matter, ·1 part per million;
For hardness, not less than 3° to 5°, and, preferably, not more than about 15° (extreme limit of 25°).

It has also been seen that, of these, temporary hardness may be removed by treatment in tanks with lime water, and organic matter may be more or less completely removed by efficient filtration, while bacteria may be abstracted to the extent of some 50 per cent. by sedimentation and by from 98 to 99 per cent. by filtration through sand. Poisonous metals require special treatment. Hardness may be added, if necessary, by lime in filters.

Hence the necessity of knowing the exact composition and history of the water becomes apparent, and the engineer in his choice of a source must be guided by both these investigations.

A water of fair composition but bad history must be looked on with suspicion. Such sources of pollution as cemeteries, undrained houses and farms, etc., in the area drawn upon, are to be avoided. Filtration may purify such waters, but it is better to attack the evil at its commencement and remove the cause of danger, lest the protection should break down without notice. On the other hand, a water of suspicious composition may, on investigation of its history, prove to be quite safe.

In general, surface waters would be first looked to as sources for towns, both because their history is generally more easily studied and because they may save the cost of pumping; but it is also to be borne in mind that pumping supplies are generally cheaper in first cost than those which deliver by gravitation, while the working expenses of the former system of supply are greater than those required to maintain the latter.

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CHAPTER III

REQUIREMENTS AS TO QUANTITY

Quantity required.—The quality of the water from a given source having been found suitable, the question of quantity is next for consideration.

The supply required for present needs is arrived at by estimating at a certain rate the consumption per head of population after deducting a percentage, which will differ in each case, for those who are already provided with an efficient supply of water, and are disinclined to pay an additional rental for the sake of the convenience of getting water delivered into their dwellings. Even in large towns there generally are many private supplies, but they are particularly abundant in rural districts. The deduction referred to above will therefore, as a rule, be greater in the country than in towns. As time goes on, and it is made manifest that the advantage of getting water for the labour of drawing it is much more than counterbalanced by that of obtaining a plentiful and probably better supply by merely turning a tap, at a cost which is so small as to be little felt, those who for a time resisted the temptation to take the water offered to them overcome their scruples or fears, and eventually the whole or very nearly the whole of the population has to be provided for.

Some persons refuse to pay for water, because they say that it is a necessary of life and free to all. So it is, for the carrying, but the transportation may mean a large expenditure of time and labour, and it is not the water which must be paid for, but the privilege of having it collected and delivered, for which a charge is made.

Rate per Head.—The rate to be allowed per head is a question about which there is some difference, but on analysis much of the difficulty disappears, and it is found that for domestic purposes, that is to say, for drinking and washing, including baths, culinary purposes, and the flushing of closets, the unrestricted consump-

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tion seldom exceeds 17 gallons per head per diem in towns and 12 gallons in rural districts, and a provision of 20 gallons per head in urban and 15 gallons per head in rural districts is ample. The use of water is not, however, restricted to domestic purposes. Trade purposes, watering gardens, steam boilers, washing in mills, cleansing vehicles, cooling in breweries, and a multitude of other industries, not to speak of the watering of streets and the flushing of sewers, have to be provided for, and the quantity of water consumed under this head varies within very large limits, from as little as 5 gallons per head to about 20 gallons a head or more. Instances occur where the trade supply, which is generally paid for at a meter rate, greatly exceeds that used for domestic purposes.

The following figures of average daily supply in gallons per head of population serve to show how the quantity varies in different localities:—

Aberdeen, . .	43	Manchester, .	27
Birmingham, .	24	Perth,. . .	50
Carlisle, . .	23·3	Plymouth, . .	43
Glasgow, . .	52	Sheffield, . .	22
Leicester, . .	16	Wigan, . . .	17
Liverpool, . .	25		

In other places the rate of supply has been as high as, for Philadelphia, 215 gallons per head per day, and for New York, 112 gallons per head per day.

These figures indicate great waste, which is due partly to the use of some of the water for pumping a smaller bulk to high levels by means of water-pressure engines, and partly to the fact that the municipal authorities fear to incur odium by the adoption of proper steps for the prevention of waste.

Until comparatively recently, water was not generally supplied continuously, but intermittently for probably three or four hours a day, and householders depended for continuity of supply on cisterns in their dwellings, which were supposed to be and in many cases were filled during the hours when the water supply in the mains was on; if, however, the supply were not afforded for a sufficient length of time, or there was a greater demand than usual for water, the cisterns at the lowest level were first filled, then those which were a little higher, and those at the highest level obtained their

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supply last, and these cisterns might be only partially filled or not at all. Another objection to a system of intermittent supply is that the mains become partially emptied, and there is a liability to the introduction of foul matter or air to the pipes, and in case of fire the water is not ready.

At the present date almost all supplies are constant. The mains are constantly charged under pressure, and cisterns, where they exist, —and no house should be without a cistern—are filled again immediately they are drawn on.

Under conditions of constant supply the only limitations on the quantity of water which householders can use is to be found in the diameter of the service pipes, in the pressure on the main, and the number of taps, which is generally considerable, yet the quantity of water used for domestic purposes referred to above is rarely exceeded on an average, though the consumption in one household may differ very largely from that of its neighbour.

Variations in Demand.—The demand for water is variable not only from year to year, but from month to month, from day to day, and from hour to hour. In hot, dry years the consumption is greater than in wet seasons. There is more water used in baths and in watering gardens and streets in dry years than in wet. Similarly, the summer consumption in periods of drought exceeds that of an ordinary winter by a greater amount for the years given above, while, should there be a severe frost and the water in the service pipes is likely to freeze, it is usual to find that the consumption of water has increased at an abnormal rate owing to taps having been left running under the erroneous impression that this would prevent freezing and the bursting of pipes.

Monthly Variations.—Taking an average of fifteen years, the supply of London derived from the Thames has varied in the following ratios :—

January,	93·6	July,	112·8
February,	91·3	August,	109·3
March,	92·1	September,	106·1
April,	94·0	October,	100·9
May,	100·9	November,	96·6
June,	108·0	December,	94·1

The highest percentage of supply was 121 per cent. in July, the lowest 85·5 per cent. in February. For a single day the demand may exceed the average by 40 per cent.

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Hourly Variations.—During any twenty-four hours the variations in the demand for water are considerable, and the incidence and amount of these variations are by no means constant; it may, however, be accepted as a broad general rule that the greatest demand, which may be from 50 to 100 per cent. above the average, will occur in the morning at and after breakfast time, that it will decrease towards mid-day, and increase again to dinner time, and will again decrease to its minimum at or about 3 a.m. or after, probably, commencing to increase at about 5 a.m.

Waste.—During the early hours of the morning there should be little water passing through the mains. If the quantity flowing is measured by an automatic recorder, should the line drawn by the pen be straight, or nearly straight, with but small variations in level here or there which record distinct demands on the supply, the quantity recorded is waste, and steps should be taken to locate and obviate it. Waste is not entirely preventible, but the greater proportion of it can and should be prevented; it is a source of loss to the public at large in two directions—it demands a greater supply of water than is necessary, and is therefore a direct money loss; it necessitates the use of larger pipes and machinery than would otherwise be requisite, which is also money lost, and it renders obligatory the use of larger sewers and sewage disposal works than would be required if there were no waste. Preventible waste is not only a direct loss to the consumer from a monetary point of view, but it also prevents his getting the water he requires so freely as he otherwise would, yet the prevention of waste is largely in his own hands.

Waste—*i.e.* preventible waste—arises from the following amongst an infinity of small causes, namely, the undetected leakage of pipe joints. Cisterns of imperfect construction are responsible for a large amount of waste owing to the valves not being firmly attached to a fixed support, to weak construction, or some similar cause. There are few houses in which one or more taps do not drip, and a dripping tap may mean the loss of as much as ten gallons a night, while the useless and pernicious waste of water in gardens to no purpose is deplorable. There are, it is believed, but very few places in which the amount of waste is less than 10 per cent., and the writer has known of cases where it has been as much as 66 per cent. of the total supply.

Future Extension.—In laying out waterworks, it is always desirable to look ahead, and to make some provision for the future. Many large towns have looked so far forward that they may be

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considered to have secured a sufficient area of supply to last for an indefinite number of years; they are, however, limited, as to their powers to borrow money, to a much shorter period, and it is usual for Parliament to look with suspicion on any claim which, unless complete in itself, does not require review in something like fifteen years' time.

If a town or district is increasing rapidly in population, the engineer should direct his attention, not only to the provision of supply amply sufficient for the immediate requirements, but should consider the future necessities of the locality, and the best means of ensuring the supply to meet demands at a reasonable cost, and for these reasons to secure a larger contributory area than that which is sufficient to provide for the immediate wants of the population.

What the increase in fifteen years, or in some other stated period, is likely to be, can be estimated approximately by studying the past rate of growth of the population, and examining any special circumstances likely to accelerate or retard it in the future.

It is to be borne in mind that the rate of increase follows, more or less exactly, a mathematical law, and in England, where the census is taken every ten years, it is generally stated at so much per cent. per decade.

Graphical Determination of Rate.—The average rate of increase may be determined arithmetically, or graphically, by plotting the past populations in a curve, the latter being drawn in an average position between the plotted points.

A more satisfactory result may be obtained by plotting the *logarithms* of the population. For, the rate being assumed regular, the population and the period of years are connected by the equation $y = a\,b^x$, where

y = population at end of any period;
a = „ at beginning of same period;
b = ratio of population at end of ten years to population at beginning;
x = number of terms of ten years in period considered.

If we take logarithms, we get $\log y = \log a + x \log b$, which is the equation to a *straight line*, so that the result will be that the plotted points will lie in most cases nearly in a straight line, and the average rate of increase can be found by drawing the line which most nearly coincides with them. This is more easily determined for a straight line than for a curved one, and can, moreover, be produced with less chance of error.

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For example, suppose the population of a town in thousands was as follows:—

Year.	1830.	1840.	1850.	1860.	1870.	1880.	1890.	1900.
Population.	20·0	23·5	29·0	34·2	41·0	49·6	57·7	68·8
Logarithm of Population	1·3010	1·3711	1·4624	1·5340	1·6128	1·6955	1·7612	1·8376

The top and bottom lines of this table have been plotted at a, b, c, etc. (fig. 3), and it will be observed that the plotted points fall nearly in a straight line.

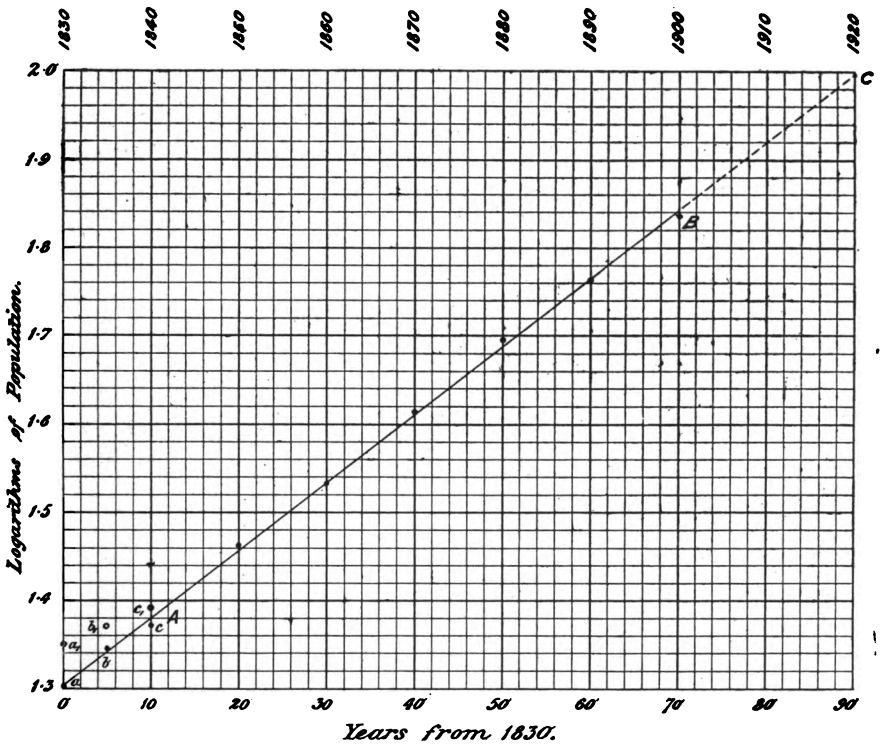

FIG. 3.

By stretching a fine thread on the diagram, the straight line A B, which approximated most closely to a mean position between them, was determined and drawn.

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This line having been determined, the probable population for any future year can be ascertained by scaling the vertical line corresponding to the year in question to its intersection with the line A B; thus the log of population for 1910 is 1·9200, corresponding to a population of 83,177 persons, and so on.

If all the points will not agree, even approximately, with a straight line, the later ones only should be used. Thus if the first three points had been a_1 b_1 c_1 they would have been neglected, and the line taken in the same position as in the case considered.

Quantity Available; how Estimated.—The different sources of water have been mentioned at sufficient length in Chapter II. The consideration of them anew, with special reference to the quantity, rather than the quality, of the water supplied by each, may now be proceeded with.

It may be said at the outset that, no matter what the source may be, the first thing to be done is to endeavour to arrive at the size of what is known as the 'catchment area' or 'gathering ground' belonging to that source, and at the average rainfall on that area.

The 'catchment area' means the surface on which the rain supplying the source falls. This is generally, in this country, expressed in acres, or even more conveniently in square miles of 640 acres.

The annual rainfall, multiplied by the catchment area, gives the volume of water falling as rain on that area per annum.

This has to be reduced to the requisite units (gallons or cubic feet) after making proper allowance for evaporation and other sources of loss to be referred to later. The remainder is the rainfall available for supply.

Factors of Reduction.—

One acre = 4840 square yards = 43,560 square feet.

One inch = $\frac{1}{12}$th foot.

One c. ft. = 6·24 galls. about.

∴ For rainfall of 1 inch on 1 acre—

$$\text{Quantity} = 43{,}560 \times \tfrac{1}{12} = 3630 \text{ cubic feet} \quad . \quad . \quad . \quad . \quad . \quad (1)$$
$$= 3630 \times 6{\cdot}24 = 22{,}650 \text{ galls.}$$

Hence to obtain the quantity, multiply the catchment area, or area of the gathering ground, in acres by the *available* rainfall in inches, and if the quantity is required in cubic feet, multiply the result by 3630; but if required in gallons, multiply instead by 22,650.

It is convenient for purposes of approximate calculation to remem-

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ber that 1 inch of rain per annum represents a flow of 40,000 gallons a day from each square mile of catchment area . . . . . (2)

Determination of Catchment Area.—If the proposed source be a stream, river, or lake, it is necessary to determine, by a survey, the 'watershed' separating the valley from which the supply is to be drawn from adjacent valleys.

Thus, if the water is to be drawn off at A (fig. 4), the area B C D

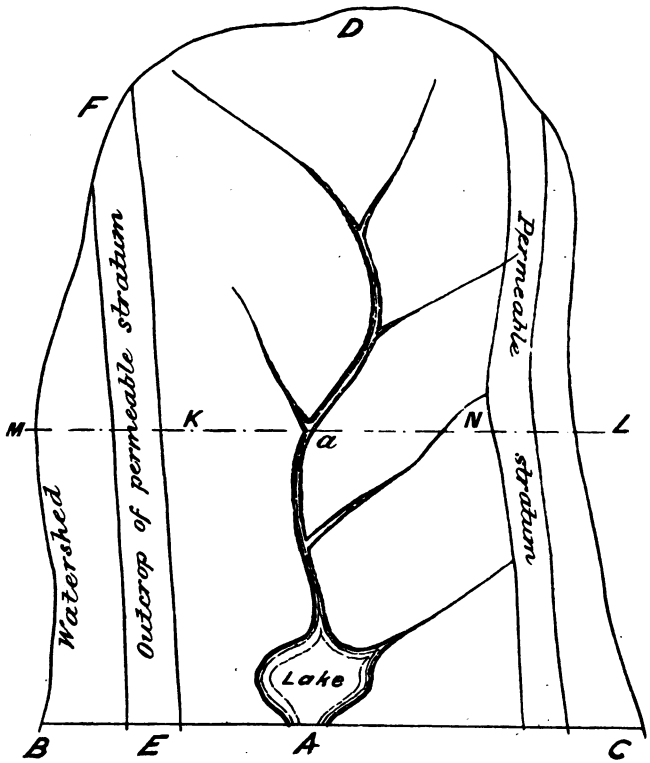

FIG. 4.

would, unless there be some hidden leakage either into or out of the valley, be the gathering ground, and its extent may be measured from a plan or map.

It should be observed that in practice the line B C is rarely at right angles to the stream, as the general trend of the land is downwards both towards the stream and along it.

There may, however, be additions or deductions to be made to this in determining the true catchment area.

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Thus, suppose on the sloping hillside there should be a layer of permeable strata cropping out at E F (fig. 4). If this layer dip from the valley as shown in section at K (fig. 5), all, or much, of the water falling on the sloping hillside, M K, instead of feeding the stream *a*, may pass through the porous layer K and so into an adjacent valley.

Similarly, if there be another porous layer on the other side, sloping towards the valley as at L, such a layer will cause water to flow into the stream which would otherwise belong to another catchment area.

A simple case is shown in the figure, but it is to be understood that, wherever the surface is at all porous (as a great deal of it probably is in every catchment area) some of the water will penetrate deeply below it, instead of flowing off over the surface. Whether this water will again reach the surface in the same valley as that in which it fell, or not, will depend upon the arrangement of the permeable and the underlying impermeable strata.

FIG. 5.

Even if the water does return to the same stream, it may do so below the point at which the supply is to be drawn off, and hence will be lost for the purpose of that supply.

To allow, with any degree of accuracy, for such gains and losses is a difficult matter, which must be estimated to the best practicable degree, and may be checked by a comparison of the rainfall in two or three adjacent valleys with the gauged flow of the streams proceeding from them.

If the district be one in which chalk beds, limestone, loose sands, etc., are abundant, it becomes particularly difficult. In many cases, in limestone rocks particularly, large fissures exist which may cause enormous quantities of water to pass underground to some other area than that on which they fell

The slope of the ground is also an important matter. On steep slopes the water will run off quickly, and the losses, both by

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evaporation and by permanent and temporary abstraction by underground percolation, will be less than on flat ground of similar character.

Similarly, if a rainfall be rapid, the percentage of it finding its way underground will be less than if the rainfall be slow.

The same thing applies to showers falling after heavy frosts when the soil is ice-bound.

The opposite is thought by some to hold with falls of snow, which melts first, in general, next the ground and not at the surface. Melting slowly there, a large percentage of it will sink into the ground; but that this occurs is disputed by many observers.

In certain cases the water thus percolating finds its way into the sea, below the surface-level of the latter.

Where springs are found rising from the sides or bottom of a valley, a knowledge of the geological formation of the district may enable a conclusion to be arrived at as to whether the water which feeds them fell within the surface drainage area of that valley or not.

Thus, if the layer L N (fig. 5) were of sufficient extent, and of sufficiently retentive material, and fed by a large enough area, a spring might rise at N, running either continuously or intermittently, and a knowledge of the arrangement of the strata would enable us to say that part of the water yielded by it belongs to another area, and must be added to the quantity calculated by the rainfall on the catchment area of the stream *a*.

Water running *away* from the valley, or running into the bed of the stream below the water level, or oozing out all over the hillsides, is liable to escape detection.

Hence it will be evident that a careful study of the surface formation of the district is necessary, as well as the determination of the apparent catchment area and the rainfall, in order to arrive at a reliable estimation of the yield.

A knowledge of geology is useful to the engineer in this respect, and the geological maps of the Ordnance Survey will, if properly understood, be of much assistance; but it must be remembered that the general geological survey maps deal with the rocks which underlie the area under observation, and afford little or no information as to the actual surface where covered by shallow drifts, gravel-beds, etc.; for information as to these beds, which frequently determine the capacity of the area for percolation, reference must be made to the superficial drift maps, which are far from being complete.

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Estimation of Rainfall.—As the amount of water to be derived from any drainage area is to some extent uncertain, it is desirable to employ all possible checks, which may be provided by the observation of several rain gauges, and their comparison with existing gauges of long standing, and with continuous gaugings of the water running from the area. The methods of gauging, and the consideration governing their use, will be referred to later.

Deductions from Rainfall. Evaporation. Seasonal Change. Flood.—Even supposing the area contributing to the source of supply to be accurately determined, the average rainfall to be known with a fair degree of accuracy, and the stream to have been gauged for a longer or shorter period, deductions must be made from the average rainfall for evaporation and for seasonal changes, there is a further loss, that due to flood, to be considered. When the reservoir is already full, if the stream should run down in flood, the whole or the greater part of the rainfall for this period is lost for purposes of supply, and this loss would be a serious item were it not that provision having to be made for seasonal changes, and the capacity of the reservoir calculated on the available rainfall of the three driest consecutive years, the reservoir is unlikely to be full, and the chance of its being unable to impound the storm water during such periods of drought is small; nevertheless, in hilly districts with a heavy rainfall it is found necessary to provide for some loss on account of unintercepted storm waters.

There is a certain amount of evaporation going on during the whole of the year, and that evaporation varies with the temperature, the humidity of the atmosphere, the amount of sunshine, and to some extent with the amount of the rainfall.

The extent of these variations is not great, probably not more than 3 inches of rain per annum. Evaporation in the south of England is generally taken as from 13 to 16 inches of rain, and in the north it varies from 12 to about 15 inches. If the rain come as a sudden thunderstorm, when the ground is parched, it may be at once absorbed, or if the ground be saturated it may flow directly over the surface to the stream, leaving little time for evaporation, whereas with a fine and continuous rain, accompanied by wind, the evaporation will be high. Another varying condition is the quantity of vegetation in the locality, the effect of which is to increase the loss as compared with bare rocky hillsides.

No definite figures for evaporation can be given in face of the

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universally varying conditions, nor can there be said to be any definite relation between evaporation and rainfall.

The average rainfall on the catchment area is estimated by means of rain gauges.

The most common form of rain gauge is that known as the Snowdon pattern (fig. 6), consisting of a japanned, or preferably copper cylinder, *a*, sunk till the lip, which must be quite level, is 12 inches above the level of the ground.

Inside the cylinder is a funnel so arranged as to collect the rain falling within the mouth of the gauge, and lead it into a bottle, *b*.

The quantity so received is measured daily, or monthly in places which are difficult of access, by pouring the contents of the bottle into a measuring-glass, *c*, the divisions on which are calculated to

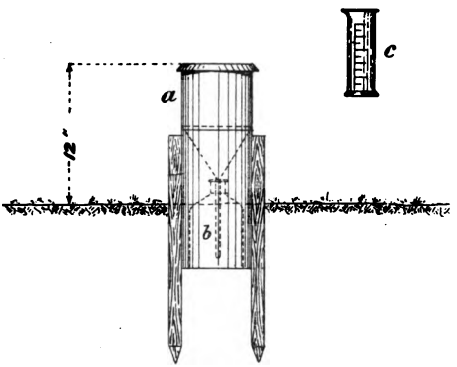

FIG. 6.

show the depth of rain which fell over the larger area of the gauge itself.

As a general guide to the observation of rainfall we here quote *in extenso* the directions issued by the late Mr J. G. Symons to the observers under his organisation, and which cover the whole ground.

SUGGESTIONS FOR SECURING UNIFORMITY OF PRACTICE AMONG RAINFALL OBSERVERS.

I. *Site.*—A rain gauge should not be set on a slope or terrace, but on a level piece of ground, at a distance from shrubs, trees, walls, and buildings—at the very least as many feet from their base as they are in height. Tall growing flowers, vegetables and bushes must be kept away from the gauge. If a thoroughly clear site cannot be

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obtained, shelter is most endurable from N.W., N., and E., less so from S., S.E., and W., and not at all from S.W. or N.E.

II. *Old Gauges.*—Old-established gauges should not be moved, nor their registration discontinued, until at least two years after a new one has been in operation, otherwise the continuity of the register will be irreparably destroyed. Both the old and the new ones must be registered at the same time, and the results recorded for comparison.

III. *Level and Fixing.*—The funnel of a rain gauge must be set quite level, and so firmly fixed that it will remain so in spite of any gale of wind or ordinary circumstance. Its correctness in this respect should be tested from time to time.

IV. *Height.*—The funnels of gauges newly placed should be 1 foot above grass. Information respecting height above sea level may be obtained from the Editor.

V. *Rust.*—If the funnel of a japanned gauge becomes so oxidized as to retain the rain in its pores, or threatens to become rusty, it should have a coat of gas tar, or japan black, or a fresh funnel of zinc or copper should be provided.

VI. *Float Gauges.*—If the measuring rod is detached from the float, it should never be left in the gauge. If it is attached to the float it should be pegged or tied down, and only allowed to rise to its proper position at the time of reading. To allow for the weight of the float and rod, these gauges are generally so constructed as to show 0 only when a small amount of water is left in them. Care must always be taken to set the rod to the zero or 0.

VII. *Can and Bottle Gauges.*—The measuring glass should always be held upright; the reading is to be taken midway between the two apparent surfaces of the water.

VIII. *Time of Reading.*—Nine a.m. daily; if taken only monthly, then 9 a.m. on the 1st.

IX. *Date of Entry.*—The amount measured at 9 a.m. on any day is to be set against the previous one; because the amount registered at 9 a.m. of, say, 17th, contains the fall during fifteen hours of the 16th, and only nine hours of the 17th. (This rule has been approved by the Meteorological Societies of England and Scotland, cannot be altered, and is particularly commended to the notice of observers.)

X. *Mode of Entry.*—If less than one-tenth (·10) has fallen, the cypher must always be prefixed; thus, if the measure is full up to the seventh line, it must be entered as ·07, that is, no inches, no

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tenths, and seven hundredeths. For the sake of clearness, it has been found necessary to lay down an invariable rule that there shall always be two figures to the right of the decimal point. If there be only one figure, as in the case of one-tenth of an inch (usually written ·1) a cypher must be added, making it ·10. Neglect of this rule causes much inconvenience. All the columns should be cast *twice*—once up and once down, so as to avoid the same error being made twice. When there is no rain, a line should be drawn rather than cyphers inserted.

XI. *Caution.*—The amount should always be written down before the water is thrown away.

XII. *Small Quantities.* — The unit of measurement being ·01, observers whose gauges are sufficiently delicate to show less than that, are, if the amount is under ·005, to throw it away; if it is ·005 to ·010 inclusive, they are to enter it as ·01.

XIII. *Absence.*—Every observer should train some one as an assistant; but where this is not possible, instructions should be given that the gauge should be emptied at 9 a.m. on the first of the month, and the water bottled, labelled and tightly corked, to await the observer's return.

XIV. *Heavy Rains.*—When very heavy rains occur, it is desirable to measure immediately on their termination, and it will be found a safe plan after measuring to return the water to the gauge, so that the morning registration will not be interfered with. Of course, if there is the slightest doubt as to the gauge holding all that falls, it must be emptied, the amount being *previously* written down.

XV. *Snow.*—In snow three methods may be adopted—it is well to try them all. (1) Melt what is caught in the funnel by adding to the snow a previously ascertained quantity of warm water, and then deducting this quantity from the total measurement, enter the residue as rain. (2) Select a place where the snow has not drifted, invert the funnel, and turning it round, lift and melt what is enclosed. (3) Measure with a rule the average depth of snow, and take one-twelfth as the equivalent of water. This being a very rough method, is not to be adopted if it can be avoided. Some observers use in snowy weather a cylinder of the same diameter as the rain gauge and of considerable depth. If the wind is at all rough, all the snow is blown out of a flat-funnelled rain gauge. Snowdon pattern gauges are much the best.

XVI. *Overflow.*—It would seem needless to caution observers on

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this head, but as not a year passes in which some of our observers do not allow it to happen, it is necessary to call attention to the fact that there does not seem to be any part of the British Isles where 4 inches will not fall in twenty-four hours. Therefore it is not desirable to purchase any new gauge of which the capacity is less than 4 inches.

XVII. *Second Gauges.*—It is desirable that observers should have two gauges, and that one of them should be capable of holding 8 inches of rain. One of the gauges should be registered daily, the other weekly or monthly as preferred, but always on the first of each month. By this means a thorough check is kept on additional errors in the entries, which is not the case if *both* are read daily.

XVIII. *Dew and Fog.*—Small amounts of water are at times deposited in rain gauges by fog and dew; they should be added to the amount of rainfall, because (1) they "tend to water the earth and nourish the streams; and not for that reason only, but (2) because in many cases the rain gauges can only be visited monthly, and it would

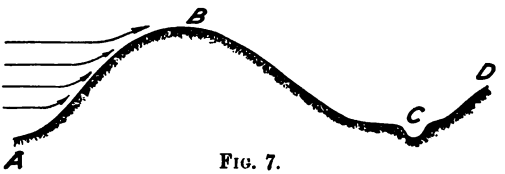

FIG. 7.

then obviously be impossible to separate the yield of snow, rain, etc.; therefore, for the sake of uniformity, all must be taken together."

XIX. *Doubtful Entries.*—Whenever there is the least doubt respecting the accuracy of any observation, the entry should be marked with a ?, and the reason stated for its being placed there.

Variation of Rainfall.—There are very few districts of any size in which the annual rainfall is everywhere the same. A wind blowing against the side of a hill in one place may be thereby compressed, while in another place, blowing over the tops of the hills into the next valley, it may expand immediately after crossing the ridge, and thus have its vapour-carrying capacity diminished. Thus a rain-bearing wind blowing in from the coast A will expand suddenly on crossing the ridge B, and will deposit more of its moisture on the side B C of the next valley than will be deposited on the side C D. The rainfall on B C will be greater also than that on A B, unless the range of hills is very high.

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In this case the maximum rainfall is on the windward side, or on the side A B in the figure.

Causes of Variation.—The quantity of water vapour that the air can contain, in fact, depends upon the temperature and the pressure. Hence if the air be saturated with vapour, any circumstance tending to decrease its pressure or its temperature will probably cause an immediate deposition of some of the moisture. On the other hand, any circumstance tending to increase either of these will tend to diminish the tendency to deposit moisture.

When a wind blows against the side of a hill, it is compressed, and this increases the pressure. But it is also deflected into the higher regions of the atmosphere where the pressure and the temperature are both less.

Hence there are conflicting tendencies at work, and the sum total, together with the degree of saturation of the air, will decide the result in each case.

For much the same reasons, the rainfall generally increases with the elevation; and a common rule, based principally on observation, is, in England, to allow from 2½ to 3 per cent. of the rainfall per 100 feet of elevation above the gauge. In wet districts the addition is less, and is often taken at 1 inch per 100 feet.

This rule, however, has exceptions, as may readily be imagined. Thus the rainfall at A (fig. 7) may be greater than at D, though D is higher up, on account of the moisture which has already fallen, leaving the air in a drier state.

In England the highest recorded rainfall is at Seathwaite, in Cumberland. There is a long valley extending in a south-westerly direction to the sea, with a narrow pass between two high mountains at its head.

The rain-clouds coming from the sea are gradually compressed as they pass up the valley, till, on surmounting the pass at the head, they are free to expand, and in so doing deposit the moisture which they contain on the further, or north-eastern, slope of the hills at Seathwaite.

It is to be remembered, in this connection, that the greater part of the moisture in the air is taken up in tropical latitudes. Hence the moist winds in the temperate and frigid zones will be those which blow from the tropics; that is, in England the south-west winds.

The same things we have been explaining take place on a smaller scale in the different parts of each district.

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Hence it is clear that one rain gauge in a district would afford a most unreliable idea of the general rainfall over that district.

Snow.—Another difficulty arises in the case of a heavy fall of snow. This may either be blown away from over the mouth of the gauge, or it may melt more quickly over the gauge, and form a sort of inverted funnel, causing a much greater record than is warranted by the true fall.

In order to show the effect produced by circumstances such as those we have been considering, a list of average rainfalls in inches per annum is here given for different places :—

Penzance,	42·76	Ely, Cardiff,	43·60
Ilfracombe,	30·63	Glyncorrwg, Neath,	87·50
Southampton,	31·55	Builth,	67·17
Reading,	25·86	Carmarthen,	51·70
Brighton,	28·33	Haverfordwest,	50·36
Greenwich,	24·94	Aberystwyth,	54·60
Dover,	27·72	Chester,	28·43
Brentwood,	26·06	Manchester,	37·42
Yarmouth,	27·58	Preston,	34·73
Cambridge,	23·19	Clitheroe,	43·04
Hertford,	25·89	Barrow-in-Furness,	38·94
Tring,	29·90	Monk Coniston,	78·68
Northampton,	24·85	Kendal,	50·34
Loughborough,	25·14	Kirkby Stephen,	39·95
Retford,	23·43	Rotherham,	22·39
Buxton,	54·02	Redmires, Sheffield,	42·12
Derby,	24·35	Keswick,	60·33
Oxford,	26·15	Seathwaite,	136·04
Clifton,	35·44	Lincoln,	23·63
Leamington,	26·25	Hull,	27·86
Ross,	30·89	York,	24·84
Stourbridge,	27·08	Richmond, Yorks,	34·05
Ludlow,	31·03	Darlington,	25·90
Chepstow,	35·54	Newcastle,	29·20

Variation with Time.—Hence to obtain the true rainfall all over a district, as many gauges as possible must be used situated in places representative of the different elevations and conditions to be met with in the district.

Besides the variations between places between days and months, there are also yearly variations; and in order to arrive at a true average, it is necessary that gauging should be taken for at least thirty-six years.

Comparison of Results.—As, however, an approximation to the true average must be arrived at in a much shorter period, a fairly

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satisfactory result may be obtained from the observation of several rain gauges for some months and comparing the observation with the records of gauges which have been in existence for thirty-six years or more.

The rainfalls, as registered by the different gauges, will probably bear much the same ratio to one another during the period of observation as they will during a longer period. If, then, somewhere in or near the district, some one or more gauges can be found the records of which have been kept for thirty-six years or more, and the records of all the gauges be compared during the time of co-existence, a ratio can be arrived at which will give, with considerable accuracy, an average for each of the newly-instituted gauges.

Thus, suppose rainfall by the new gauges in six months = 11·7 inches, that by standard gauge in the same six months = 9·89 inches, and average for thirty-six years by standard = 23·4 inches per annum. Then rainfall by our gauge : that by standard gauge : : 11·7 : 9·89.

∴ average annual rainfall at the place where our gauge is placed

$$= \frac{11{\cdot}7 \times 23{\cdot}4}{9{\cdot}89} = 26{\cdot}26 \text{ inches nearly.}$$

If no record can be found which covers the whole period, there may be several each of which has been kept for some shorter term of years, these periods more or less overlapping one another. Then a comparison of the records where they overlap will enable us to fill in, as well as we can, in a similar manner to that described above, the complete record for one or the whole of the gauges.

In this way a conclusion as to the average annual rainfall that would be registered by each of the newly-instituted gauges during a long period may be arrived at.

If the gauges be equally spaced over the area, an average referable to each elevation may be obtained and a general average arrived at. If, however, the gauges be not equally spaced, the record for each gauge must be multiplied by the area which it represents. This may be found by joining the gauge to all the adjacent gauges and drawing perpendiculars through the middle points of these lines to intersect with one another.

Deductions for Dry Years.—Thirty-six years is given as a suitable cycle to consider, because it appears that wet and dry periods alternate, and that the number of years referred to is likely to include at least two full cycles, in one of which the average rainfall would be above and in the other below the average; thus, taking the Greenwich records for the thirty-six years from 1865 to 1901 the average

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for the first period of eighteen years is 26·03 inches, for the second period of eighteen years is 22·36; the general average for eighty-six years having been 24·20 inches.

A complete period of wet or dry years is found in England to occupy from about eleven to nineteen years. Hence thirty-six years will about represent two or three such complete periods, and may be taken as giving nearly equal periods of wet and dry. Nevertheless, the records should be examined with a view to finding out, if possible, whether one side or the other predominates.

Records in various parts of the world show that the variation in rainfall may be expressed nearly as follows:—

Periods of three consecutive dry years may occur, the average rainfall in which is about 20 per cent. below the average, while single dry years may occasionally occur in which the rainfall is as low as 60 per cent. of the average, though there is not any reliable case in the United Kingdom where the record for three or more contiguous gauges has shown the average of three dry years to be less than 79 per cent. of the average, or for one year 66 per cent.

Reservoir Capacity.—If the full advantage of any drainage area is to be secured, the water which is in excess in the winter and in heavy storms of rain must be impounded for use in the summer, and that of wet seasons for consumption in dry ones, and the requisite storage will be found from the formula $S = \frac{500}{\sqrt[3]{R}}$, where S is the number of days' storage of the average consumption, and R is the *available* rainfall on the average of the three consecutive driest years in inches. As already explained, the available rain is arrived at by taking four-fifths of the average, and deducting therefrom the evaporation of the district on the average of similar years.

Put into figures, when the average rainfall is 25″ and 60″ respectively—

The average rain will be .	25·00″	60·00″
Deduct one-fifth for 3 dry years,	5·00	12·00
	20·00	48·00
Deduct for evaporation, say	14·00	14·00
Available rain, . . .	6·00″	34·00″

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The number of days' storage required will be in the two cases instanced 275 and 154 days respectively.

Supply direct from Stream.—If, on the other hand, it is desired to obtain a supply from a stream without storage, the available quantity will be limited by the flow of the river in question on the driest day of the driest year, a figure which cannot be calculated from the rainfall, but will probably be represented by a run off of 0·1 cubic foot per second from each square mile of drainage area, or 53·914 gallons per mile per diem, but may be less than this amount.

Example of Storage.—It may be well to quote an example so as to make the effects of storage and the advantages obtainable thereby perfectly intelligible.

Given a suitable catchment area of 2 square miles in extent, the average rainfall in which is 35 inches and the average evaporation 15 inches of rain, the available rainfall is 13 inches, the available supply for all purposes on the basis of these figures will be 1,040,000 gallons a day (*vide* (2) on p. 41). The storage capacity required will be 213 days' supply, or 221,520,000 gallons.

Provided there is no loss by floods not intercepted and stored, the storage capacity is also available for supply during the three dry years considered, and the available daily supply is increased by a fraction in the proportion of $\frac{213}{3 \times 365}$, or from 1,040,000 to 1,242,300 gallons a day; and in the same manner, if the storage be very largely increased, the average rainfall may be made available for purposes of supply, but it is seldom that to adopt this course would be advantageous, owing to the great cost involved.[1]

Gauging Streams.—The total loss of rainfall from all causes may be measured with approximate accuracy by gauging the flow of the stream itself, and comparing it with the run off which would result if all the rain which fell in the catchment area in a given time were available.

There are many ways of doing this, but perhaps the simplest is by building a dam or weir across the stream, fitted with a 'notch,' as shown in fig. 8.

The dam is generally constructed of planks, and the 'notch' is

[1] Owing to the fact that the proportion of rainfall evaporated is greater with a small than a large rainfall, it is undesirable to rely on obtaining a supply more than would be afforded by the rainfall of the average of the three driest consecutive years, unless the average rainfall is so high as to render the evaporation practically constant.

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formed in an iron plate screwed thereto, and may be either rectangular or V-shaped.

The dam and the face of the notch must be vertical, and the clear fall from the bottom of the notch to the surface of the water below should not be less than one and a half times the 'head' above the 'notch.'

The dam should be carried to a sufficient height to form a small pond above it, whose sectional area, compared with that of the

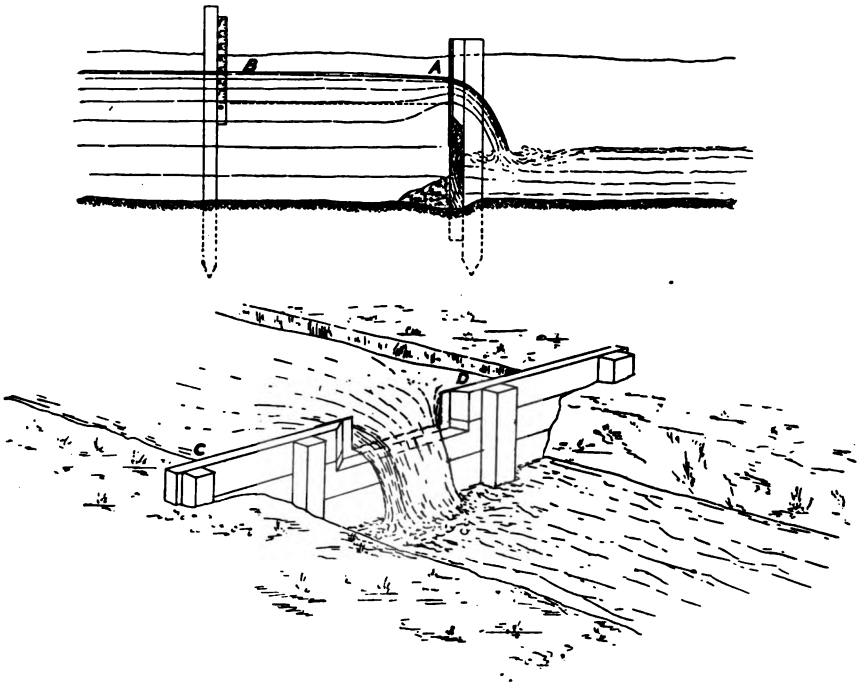

FIG. 8.

notch, is great enough to reduce the velocity of the flow through the pond practically to zero.

Near the notch the velocity will again become appreciable, and the surface will fall, so that right in the notch at A the water will not stand at as high a level as it does higher up, say at B.

The head above the notch must be measured to 'still water'; that is, it must not be measured in or near the notch.

If the length of the dam be three or more times that of the notch, it may be taken that still water will be found at the corners

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C and D, and a bracket may be placed there on the back of the dam at the same level as the bottom of the notch, the height being then measured above this bracket.

If an ordinary scale is used it must be rubbed with French chalk or some similar preparation to prevent capillary action.

In other cases the head may be measured conveniently by an anchored float which carries a scale, the latter rising and falling in contact with a fixed index whose level is known. Or the float may carry the index.

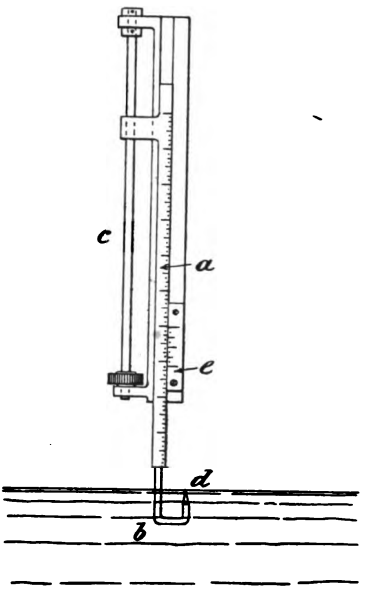

FIG. 9.

For very exact measurements a 'hook gauge' worked by a screw is used, as shown in fig. 9.

In this gauge, which is a most accurate instrument for its purpose, the divided scale *a* has attached to its lower end a hook *b*. This hook being submerged, is raised by turning the screw *c* and thereby lifting the scale till the point of the hook is just visible at the surface of the water *d*. The reading of the scale is then taken by the vernier *e*.

Before commencing operations the reading on the scale must be noted when the point of the hook is lowered until it is exactly level with the sill of the notch.

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In important cases, where a continuous record is required, a well should be sunk in the bank of the stream at sufficient distance behind the weir, and connected with the water in the stream by means of a pipe, so that the water in the well shall always stand at the same level as in the stream, and be sheltered from chance disturbances due to wind or eddies.

A float in the well is easily connected to an automatic recording clock in such a way as to give a diagram upon a revolving drum, showing at any time the level at which the water stood. The float mechanism must of course be adjusted to show zero on the diagram when the water is level with the sill of the notch.

The stream of water issuing from the notch should spring clear of the dam, and should be in contact with the notch only on the up-stream edge. For this purpose it is essential that air should have free access to its under surface on the down-stream side of the dam.

Formulæ.—Theoretical investigation shows that the discharge in cubic feet per second through such a notch should be given by the formula—

$cA\sqrt{h}$, where

A = area of notch in square feet up to still water level,

h = head above notch, in feet,

and c is a constant number depending in part upon the shape of the notch and including a coefficient due to friction and contraction, which is obtainable only by experiment.

Rectangular Notch.—Francis investigated the value of this coefficient, and determined that the discharge from a rectangular notch constructed as above may be found from the equation—

$$Q = 3{\cdot}33\ h^{\frac{3}{2}}\left\{l - {\cdot}2h\right\}$$

Where Q = discharge in cubic feet per second,

h = head in feet,

l = length of notch in feet.

Reduced to gallons per day this gives—

$$1{,}795{,}000\ h^{\frac{3}{2}}\left\{l - {\cdot}2\mathrm{h}\right\}$$

V Notch.—For a V notch the equation is—

$$Q = 1{\cdot}32nh^{\frac{5}{2}},$$

where h is the head from still water to the bottom of the V, and n is the ratio of the breadth of the V at top to its depth.

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In practice the angle A C B (fig. 10) is generally a right angle, when $n=2$, and the formula becomes

$$Q=2{\cdot}639h^{\frac{5}{2}}.$$

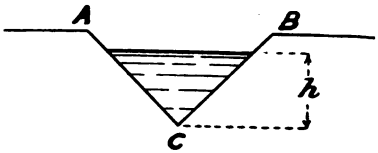

FIG. 10.

Modified Notches.—There are many modifications of the notches thus briefly described, but it is probable that the forms mentioned will be sufficient in most cases.

For streams whose flow is very variable a V notch is convenient, and it may have an obtuse angle at the vertex, n in the formula being made equal to, say, 3 or 4. Or a stepped notch, as shown in fig. 11, may be used, such that when the flow is small only the lower part will be in use, while for a large flow the wider portions of the notch also come into play.

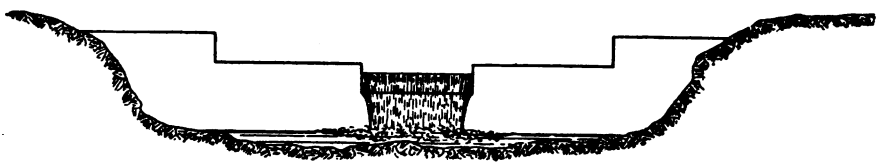

FIG. 11.

Determination of Velocity.—For larger streams or rivers the velocity of flow may be measured directly without obstructing the course.

This is done either by floats or by current meters.

The principal difficulty lies in the fact that the velocity is not the same at all points on any cross section, but is greatest at the point of greatest depth, and either at the surface or a little below it. It diminishes thence towards the sides and bottom, and some care is required in obtaining a true mean.

Whichever method is adopted, the part of the stream selected should be as nearly as possible uniform in cross section, free from weeds and other obstructions or irregularities, and straight both in the length selected for experiment and immediately above and below it, so that the 'stream lines' will be approximately parallel to the banks.

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The length, if floats are to be used, should be as great as compatible with these conditions, but they are not to be sacrificed for extra length.

Floats.—Floats are classified into surface, subsurface, twin, or rod floats, common forms of which are shown respectively in figs. 12, 13, 14, and 15, the whole being ballasted to float in still water in the positions shown.

Almost anything which will float about two-thirds or three-fourths

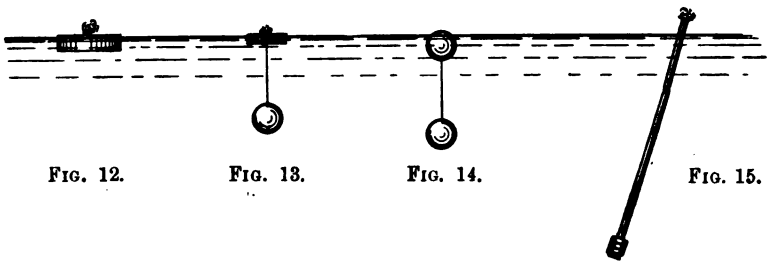

FIG. 12. FIG. 13. FIG. 14. FIG. 15.

immersed will answer as a surface float, though not more than one inch should project above the surface, to avoid errors due to winds. Subsurface floats should be put in far enough up stream to sink to the full depth of the cord before the observations are commenced. If properly constructed they will sink slowly, as the true float at the top can then be made very small and will exert little influence. Twin floats are intended to give the mean between the surface

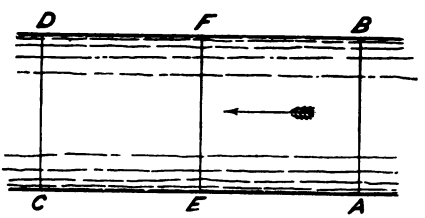

FIG. 16.

velocity and that at the depth of the lower float. Rod floats give the general mean for the whole depth of the rod, and are very convenient if the stream is regular and clean enough to admit of their being made almost the whole depth; but this is seldom practicable.

Method of Use.—The process consists in ranging out two cross sections A B, C D at a known distance apart, putting in the float up-stream of both, and noting by means of a stop watch the instant

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at which it crosses each line. Different methods of doing this accurately will occur to each observer under the conditions of the case, and several observations should in any case be taken.

The cross sections are conveniently ranged out, if the width will allow, by wires stretched across the stream, and, if the observations are to refer to different depths and different distances from the centre, these wires should be divided into corresponding sections by tags attached to them and visible from the banks, the section through which the float passes at each end being noted, and its position at the central cross section E F determined therefrom.

Working in this way a fair mean velocity at this section may be arrived at, which multiplied by the area of section gives the volume of the flow.

Current Meters.—Current meters generally consist of a small screw rotated by the current, and fitted with a counting apparatus to register the number of revolutions. These are either registered directly and the velocity determined from a simple formula supplied by the makers, or the graduation is adjusted to read the flow (in feet) directly.

They require to be tested from time to time by being towed at known rates either in still water or alternately with and against a steady current.

There are many additions to these instruments to increase facility in handling and accuracy of result. These cannot be entered into here, but one of the best forms is that made by Messrs Amsler.

Another form of current meter is that devised by Darcy, a modification of the Pitot tube, in which there are two vertical tubes, one with its end opening at right angles to the direction of flow, and the other with its end turned horizontally so as to be opposed to the current. The difference of level in the tubes is a measure of the velocity; this meter is, however, difficult to observe.

Observations being made at many points, equally spaced vertically and horizontally on any cross section, a mean may be arrived at by plotting on squared paper or otherwise.

Use of Results.—The gaugings cannot, of course, be continued for the full period of thirty-six years that has been suggested as necessary, any more than the rainfall observations; and hence, to get the available average, comparisons must be made similar to those described for rainfall.

Now, suppose the gaugings were taken for one full year, and the

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rainfall in the same time was also known. It does not follow that the flow of the stream corresponds with this rainfall, inasmuch as the rain takes some time to reach the stream. Hence the rainfall observations should commence and end before the gaugings. *How long* before is a question the answer to which depends entirely on circumstances, and cannot be exactly settled except by a long series of observations and comparison in each special case. It is often taken at from one month to three, but it is safer to commence the rainfall observations earlier and continue them later, and compare carefully with the rise and fall of the stream.

In this way a ratio between the flow of the stream and the rainfall is arrived at, and this ratio must be applied to the average available rainfall of the three successive driest years (arrived at as described on p. 52), to get the available yield.

Wells, Estimating Yield from.—We have, lastly, to consider the case where the proposed source is an underground one.

The catchment area can be determined, if the arrangement of strata be known, as already intimated, by finding the area of the outcrop of the permeable stratum, together with that of any overlying ground which feeds it, such as M K in fig. 5.

The estimation of the available rainfall has been also indicated in speaking of the deductions to be made from surface yields on pages 44 to 52.

A check on the calculation is afforded by taking the difference between the rainfall on the whole area, after allowing for evaporation and the gaugings of the streams so far as they are fed by surface waters only, as already mentioned.

Level of Water.—It is to be remembered that the surface of the water is not level, but has a slope in the direction of 'flow' (if we may so call the passage of the water through the pores of the ground). This slope is greater after wet than after dry seasons, as the head and the velocity are increased.

The actual gradient varies with the distance to be travelled and the material. In chalk or close sandstones it is on the average from 1 in 150 to 1 in 200 about; in gravel or loose sand it is somewhat less steep, and in close sand it is steeper.

Where there are fissures in the rocks the water runs through these, whereby the resistance may be lessened, and the gradient may become flatter.

The direction of flow is generally, though by no means always,

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more or less perpendicular to the direction of the valley. The depth of the line of saturation may be little or much below the surface of the ground. This depth varies with the rainfall and other circumstances (*vide* p. 42). The extreme range in many English wells is as much as 80 to 100 feet or more. In cases where the permeable stratum is at the surface of the ground, and undulating, the line of saturation may rise above it in places at wet seasons, forming a series of small lakes the surface of each of which is practically level, there being little resistance to flow above ground, but each lake a little higher than the one below it in the direction of flow, owing to the reduction in head required to force the water through the intervening ground.

The rain which feeds a given underground source will take some time to reach that source, the length of time varying very greatly

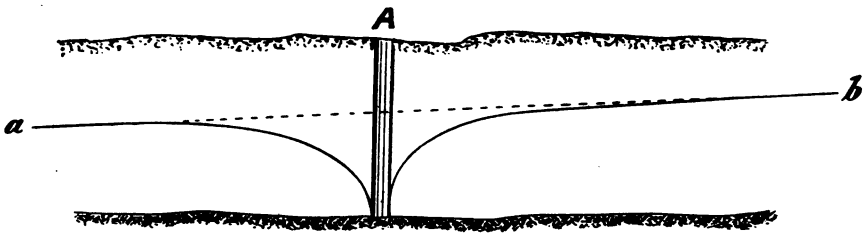

FIG. 17.

with the circumstances of the case, but may be as much as six months.

In chalk districts in England, in many cases where the rainfall begins to increase in September, the level of the underground water may continue to fall until December, when it begins to rise slowly, standing perhaps at its highest in June, in consequence of heavy rains in February.

But these rises and falls depend upon the nature of the rainfall, the amount of snow and frost, evaporation, and other circumstances, as has already been pointed out.

Drainage Area of Well.—Referring to fig. 17, if *a b* represent the original line of saturation, suppose a well at A sunk to the bottom of the water-bearing stratum, and assume that the water can enter with freedom, only limited by friction, at the sides, and that pumps of unlimited capacity are placed in it. The most they can do is to empty the well, and keep it empty.

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But the water does not run in freely, as it would if it were being pumped, say, from an open lake; but, in consequence of the resistance of the material, it runs in from all sides at a definable rate, depending, for a given resistance, upon the head of water above the bottom of the well. Hence it will be understood that increasing the capacity of the pumps beyond what will keep the well empty will not increase the discharge from the well.

It is also evident that to withdraw the whole of the water entering into any such bed as we have been considering would be difficult, and by no means an economical attempt to procure a supply. The water drawn from an underground source seldom exceeds 25 per cent. of what enters it.

The distance at which the artificial gradient brought about by the pumping will be apparent depends on the nature of the material and the depth to which the water is pumped. In sand the gradient may

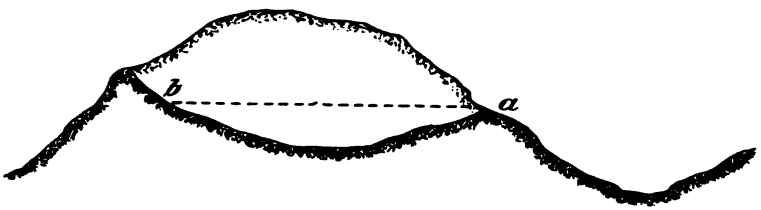

FIG. 18.

be on the average from 1 in 50 to 1 in 20, and in chalk as steep as 1 in 10; in sandstone and limestone the water generally runs through open fissures, hence the gradient is less steep. A gradient of 1 in 10 would mean that the effect of the pumping from the well would be perceptible at a distance from the well ten times as great as the depth.

Forms of Underground Reservoirs.—It will be apparent that in a case such as that shown in fig. 5, the water is retained in the porous stratum entirely by the resistance of the latter to its free passage, although the underground storage capacity is almost always sufficient to enable the water which it is possible to obtain to be taken out without restriction as to dry or wet seasons, but obviously the well cannot at all times be pumped to its full capacity and give at the same time an equal yield.

In many cases natural reservoirs occur underground. One form is shown in fig. 18, where the strata are curved.

It is evident that the water cannot run out at *a* until its level

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rises above the line *a b*, and if the well be sunk below this, we will obtain in consequence a greater head than that due to the mere dry weather flow. So long as there is any flow, the level will be *above a b.*

Another simple case arises where there is a 'fault' or displacement of the strata, such as shown in fig. 19. Faults may be either 'upthrows,' where a part of the water-bearing stratum is raised, as in fig. 19, or 'downthrows,' where it is lowered. The outlet in the original direction may be completely blocked, causing the water to rise until it overflows perhaps in some totally unexpected place, sometimes at the top of the watershed. Faults, in fact, may take an infinite variety of form, and are of much importance in waterworks engineering.

General Summary.—In testing the efficiency of a proposed

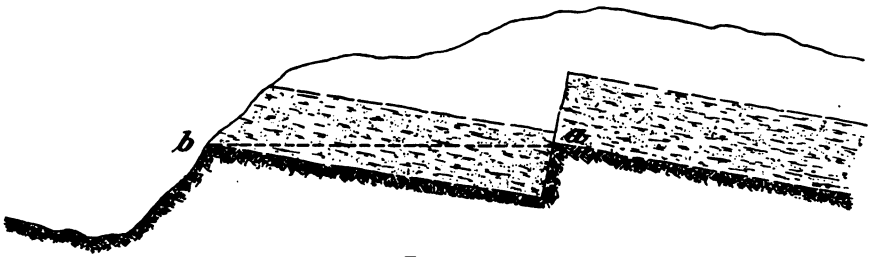

FIG. 19.

source as regards quantity, then, the annual supply required is first calculated, allowing from 20 to 25 gallons per head per day, according to the nature of the town or district (unless there are special circumstances, such as a large trade demand, to be considered), and estimating the probable increase of population for a number of years ahead, usually 15. It is generally desirable that the selected source should be capable of providing the supply likely to be required for, say, fifty years, but the works should not, as a rule, be designed to entirely meet the necessities of the case for a longer period than some fifteen years.

The amount required being thus settled, the quantity available from the source must be investigated by a careful determination of the contributing area and the average annual rainfall during a period which should, if possible, be not less than thirty-six years.

If the source be a stream, its flow should be gauged during as long a time as practicable, it being often desirable to commence the

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gaugings at a very early stage, whilst other preliminaries are being settled, and the flow compared carefully with the rainfall during the corresponding period.

A deduction of 20 per cent., or one-fifth, must be made from the average rainfall to obtain the mean fall of three successive dry years, and a further deduction for evaporation, some 12 to 15 inches, according to locality. By simple proportion, the mean flow of the stream for this dry period may then be obtained from the actual gaugings, the proportion being based on the *effective* rainfall, after the allowance for evaporation has been made. A further deduction for 'compensation' water must also be taken into account (*see* page 3).

The supply, if efficient, then falls under one of the following heads:—

(*a.*) Gravitation supplies.	(1) Where the *dry weather* supply more than meets the requirements. (2) Where it does not, but storage from wet seasons will supply the deficiency, the mean of the three consecutive driest years being more than sufficient.
(*b.*) Supplies requiring pumping.	These may also be subdivided in the same way as gravitation supplies.

It is proposed in the next chapter to consider supplies requiring storage, and to specify more fully the methods of arriving at the capacities necessary for the reservoirs, and the manner in which they are constructed.

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CHAPTER IV

STORAGE RESERVOIRS.

In perhaps the majority of cases, and certainly wherever the maximum probable demand is found to exceed the minimum flow of the stream or other source for even a very short period, some means must be devised for storing a certain quantity of water when such flow is excessive, and keeping it for use when the flow is deficient.

Requisite Capacity; how Found.—If the whole of the available supply is to be stored during the period when it is in excess of requirements, the formula on page 52 may be used. In other cases the simplest method of ascertaining the capacity of the storage required is by means of a diagram similar to fig. 20.

In this diagram the upper heavy line A B is plotted from ordinates representing the continuous addition of the available yield of the catchment basin under investigation or a similar area, expressed most conveniently in gallons, the yield during each week or month, or whatever unit of time be used, being added to the sum of the preceding units.

The lower thin line A C is plotted from a similar addition of the quantities of water required for supply and compensation during the corresponding periods, and it must be borne in mind that for reasons previously discussed, these quantities will not be equal, but will be at a maximum as a rule, at any rate in the British Isles, when the yield of the catchment area is at a minimum.

It will be apparent when the curve A B rises at a greater rate than A C, it is because the yield is rising at a greater rate than the required supply, and that, consequently, the reservoir will be filling. Conversely, when A B falls towards A C, the reverse is taking place, and the reservoir is emptying.

The amount of storage necessary may be ascertained by drawing

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another line parallel to A C from the culminating points on A B, and scaling the greatest interval between the two curves, in the example $x\ x$.

Should the yield curve fall below the supply curve at any point, as at D, it is clear that no economical amount of storage will enable the required supply to be procured from the gathering ground under consideration.

In arriving at the probable storage required by the above method, the three consecutive driest years which have been recorded should be used for plotting the 'yield' curve A B, and the greatest supply for which provision has to be made should be the basis of the 'supply' curve A C. In dealing with the three driest years, if the reservoir be assumed to be full at the commencement, the diagram is slightly altered.

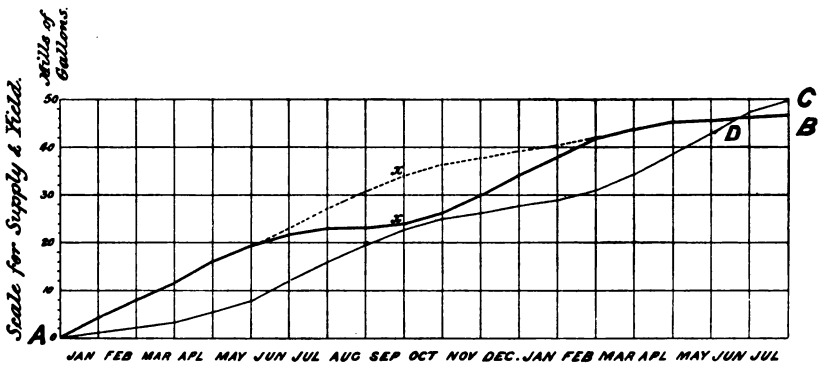

FIG. 20.

Selection of Site.—The amount of the required storage having been ascertained, the best available site next claims consideration.

The best possible site, and one which should, as far as is practicable, be sought for, is a comparatively wide and flat valley in impervious strata, preferably rock, not liable to contaminations, and ending in a narrow gorge easily blocked by a dam.

Such a site will give a maximum of storage capacity with a minimum of expense, and with practically no risk of pollution.

Subsidiary Dams and Supplies.—It is frequently desirable to impound the water yielded by other streams in valleys at a higher level than the main stream. This object may generally be effected by tapping the smaller streams at a sufficiently high level, and conducting the water round the end of the hill, or through it by means of a tunnel, into the main reservoir. By this means a considerable quan-

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tity of water may often be obtained, while avoiding a larger and more expensive dam lower down the main valley.

It may be found that the only site suitable for the reservoir has an opening in the hills surrounding it which, although at a comparatively high level, prevents the storage of sufficient water by one dam alone. In this case a smaller subsidiary dam must be employed to prevent the escape of water into the next valley.

In fig. 21 is shown a site enclosed by a main dam, A A, and a secondary dam, B B, together with a catchwater conduit, or open channel, commencing at the point C on the stream in the neighbouring valley at a high level, passing round the end of the hill, intercepting the greater part of the water running off the higher ground on its course, and finally delivering it into the main reservoir at D.

Several dams, one above the other, on the same or tributary streams, may also prove economical, some examples being the Elan Valley reservoirs for the supply of Birmingham, those near Manchester and Liverpool, and in many other places in England.

Investigation of Site.—The site having been approximately decided upon, a most careful investigation must be made of the nature of the strata underlying the place selected, as it is obvious that any fissures, or permeable beds, especially in the neighbourhood of the dam, will not only cause leakage and waste of water, but by undermining the structure itself, may possibly lead to loss of life and irreparable damage lower down the valley.

Generally speaking, the most suitable material for the bottom of a reservoir is a compact clay, or close-grained rock without fissures, and not liable to disintegrate when exposed.

Any gravel formation is evidently unsuitable unless underlaid by a considerable thickness of clay at a reasonable depth, as in the case of the Thames Valley near London.

Limestone beds form unsuitable sites for reservoirs, both on account of their fissured nature and their property of imparting hardness to the water flowing over or through them. If, however, they are covered by a layer of boulder or other clay, as is sometimes the case, these objections disappear, but care must be taken to prove that the clay is perfectly continuous and watertight, and that it extends in an unbroken layer over the whole site.

The position for the dam having been provisionally decided upon, trial pits or shafts should be sunk at frequent intervals along

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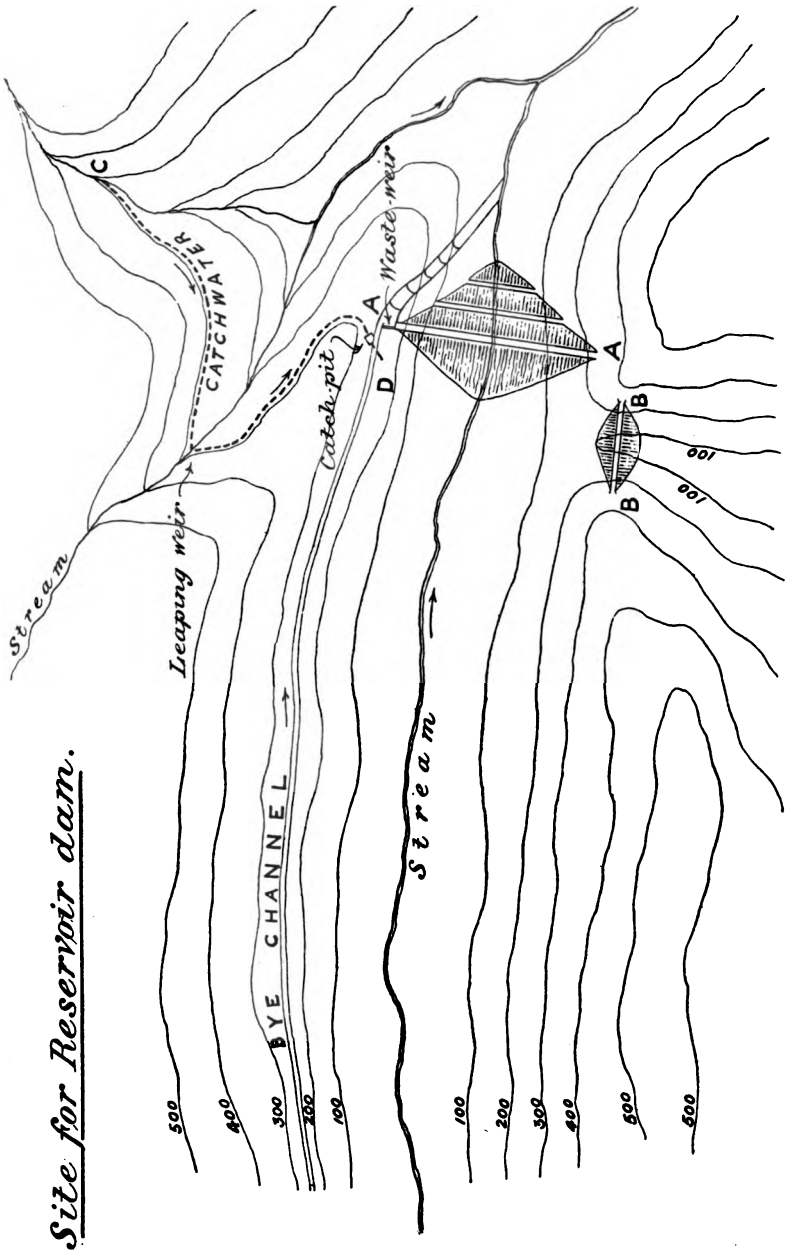
Site for Reservoir dam.
Stream
Leaping weir
CATCHWATER
C
BYE CHANNEL
Catch-pit
D
A
Waste weir
Stream
A
B
B
100
100
500
400
300
200
100
100
200
300
400
500
500

FIG. 21.

its centre line, on what, in the case of an earthen embankment, will be the puddle trench, to be hereafter described.

Large shafts are very much to be preferred to bore-holes for this purpose, as the greater area gives opportunity for the detailed examination of the strata passed through, as well as of their inclination and water-bearing qualities. A few bore-holes may be put down on the site of the main body of the dam, if it be considered necessary, care being taken to plug them securely before the work of raising the bank is proceeded with.

Requisite Height of Dam.—The site for the dam having been thus far decided upon, its height, and consequently its length, can only be settled by ascertaining the area of the reservoir site at different levels.

The most satisfactory method of doing this is by laying out contours of the whole valley, at a vertical distance apart of 5 feet, and plotting them to as large a scale as practicable. The tacheometer affords a ready means of accomplishing this work.

It is to be noticed that since the contours are 5 feet apart vertically, the *half sum* of any two areas multiplied by five, gives the volume between the corresponding contours in cubic feet.

Working in this way, not only can the necessary height of dam be ascertained, but the contents at every 5 feet, or such other interval as may be used, is accurately obtained once for all; and the amount of water coming in from the gathering ground may at all times be recorded, together with the rainfall and the quantity passing out of the reservoir to the town or district supplied, or passing to waste by the overflow or bye-wash.

These records afford a valuable means of obtaining the proportion of the rainfall which is evaporated or otherwise lost.

Area to be Acquired.—The quantity of water at each contour having thus been ascertained, that contour which represents the proposed top water level, plus a certain allowance varying with the slope of the ground, gives the minimum area of land which must be purchased. Whether it will be advisable to obtain control over any further area is a point which can only be decided by local circumstances, such as the number of human beings or animals existing upon the catchment area and liable to cause pollution. The general object, when watersheds are purchased outright, is to obtain control over the number of inhabitants and animals, and not to entirely depopulate the area.

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Some time before the reservoir is finally filled with water, all trees and underwood upon the site must be removed, and as much vegetation as possible eliminated by burning. Any dwellings or farm buildings which may be submerged must, of course, be destroyed, and all sources of pollution removed.

Earthwork Dams.—In perhaps the majority of cases, the most suitable and easily obtained material for the construction of the dam will be earth, with a watertight wall of puddled clay in the centre.

The most important part of the mass, if one portion can be said to be more important than the rest, is the puddle wall and the trench cut in the ground to receive it. This trench must be carried to a sufficient depth to ensure perfect union with an impervious stratum, and if rock should be met with, the greatest care must be exercised in stopping all fissures with strong concrete or masonry.

Any springs which may occur in the course of excavation must be led by pipes to the down-stream toe of the bank, and not held back under pressure, lest they should make their presence felt in another place, with possibly disastrous results.

Puddle Wall.—The cross section of a puddle trench is commonly made as shown in fig. 22, with a slight batter on both sides, so that any downward settlement of the puddle only tends to consolidate the material. The longitudinal section of the trench is usually somewhat as shown in fig. 23, the slopes being formed as a series of flat benches. This arrangement lends itself to ease in construction. Although the slope of the sides of the trench would appear to be detrimental, in thinning the puddle towards the bottom where the pressure of water upon it is greatest, it must be remembered that the clay itself becomes more compressed at the lower level, and thus itself compensates for the decreased thickness, and that the material in which it is set is already consolidated.

The actual thickness of the puddle wall required for any given depth of water depends so largely upon the nature of the clay or mixture of clay and stones which may be available, that no definite rule can be laid down.

It may be generally stated, however, that the minimum width at the top is 5 or 6 feet, and the thickness at the original ground level about one-fifth of the depth of water to be retained.

The clay to be used should be exposed for some time after having

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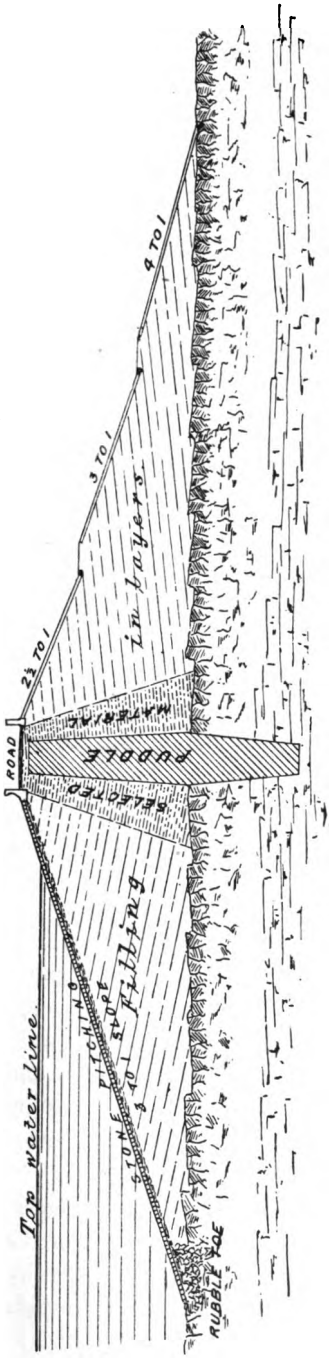

FIG. 22.

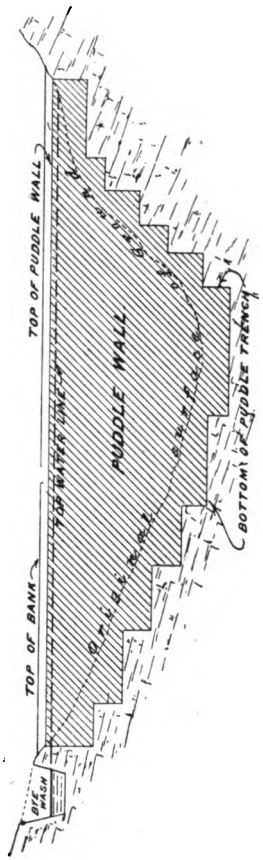

FIG. 23.

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been cut into small pieces, in order that it may disintegrate, and may then be ground in a pug-mill with a view to thorough incorporation. It is taken direct from the mill to the trench and deposited therein in layers which should not exceed 6 inches in depth, being meanwhile worked into a plastic mass by the men constantly treading it and cutting it with narrow spades or 'grafting tools.' Great care must be taken to keep the surface continually moist and irregular, so that each succeeding layer may properly unite with the one underneath. This is especially important when for any reason a considerable time must elapse before work can be resumed. It is sometimes recommended that the men treading the puddle should not wear any covering on their feet, as with bare feet they can the better detect any stones, twigs, etc., which may find their way into the trench. When the clay has been ground, however, this precaution is hardly required.

It will probably be found, especially if the puddle trench is at all deep, that pumps must be employed to keep down water which percolates in through the sides. When the trench is filled this is no longer necessary, inasmuch as the water above the wall will gradually rise to the level of the ground and will be conducted through the outlet culvert with the main stream.

Too much water is not desirable in the mixing operations, as it renders the puddle the more liable to crack when exposed to the heat of the sun.

Having arrived at ground level, it is now necessary to secure an artificial support for the puddle, which shall be carried up at the same rate, or slightly in advance of it.

Side filling.—This support is secured by what is termed the 'filling' of the embankment. A certain amount of good ballast, quarry spoil, or a clayey material if such be at hand, or at anyrate the most loamy earth which can be procured, termed 'selected material,' is tipped in layers on both sides of the puddle wall. These layers should not exceed from 6 to 9 inches in depth, and ought to be well watered while being trodden over and over by horses. Steam rolling answers very well, and is probably better than anything else so long as the roller will work.

The selected material should either be very permeable or be as nearly like the puddle as possible, in order that it may afford a good support to the puddle and may itself be well supported by the hard material forming the outer portion of the slopes, which

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is laid down in layers in the same manner and at the same time as the selected material.

If these layers are arranged so as to fall towards the puddle, the advantage is gained that any settlement tends to carry the filling towards and not away from the centre of the bank.

Before the first layer is started, the surface soil must be removed to a depth of 9 inches or a foot as may be found necessary.

The soil is reserved for the purpose of covering the outer slope of the bank when completed.

The inclination of the surface of the bank is usually made 3 to 1 on the inner or water side, and 2 or 2½ to 1 on the outer slope.

It is sometimes advisable, if the bank is of considerable height, to form one or more flat surfaces or benchings on the outer slope. These benchings prevent, to a great extent, the destructive scouring of the surface soil due to heavy rains, especially before the turf has had time to become dense. They also serve to distribute the pressure on the bank over a greater area, an advantage when the foundation is at all doubtful, and danger from slipping is to be provided against.

The slope is sometimes varied, being made flatter at the base of the embankment than at the top, the change of inclination being made at each benching.

An open drain of stoneware half-pipes may with advantage be provided at the foot of each slope, to catch the water running off and to conduct it to the stream.

Width of Dam.—The width of the bank at the top depends upon the height and the weight of the material, but would not as a rule exceed 15 or 16 feet, unless a road were required along the bank, in which case the width would necessarily be determined by that of the road, in its turn depending upon the amount of traffic expected. Such a road is very frequently required by landowners or local authorities, as a substitute for another road or roads crossing the site.

Forming Slopes.—The outer slope of the bank is usually covered with good soil and turfed, or sown with grass seed, preferably the former.

The inner slope, which is subjected to the action of the water, is covered by a layer of 'pitching' of roughly squared stone laid dry, with the interstices blinded by thin slips of the same stone. Under the pitching, which should be about 18 inches in thickness

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for a reservoir of considerable size, a layer of clay or gravel, bedded in clay, should be placed, not with any intention of keeping water from the bank, but to prevent the material of which the latter is composed from being washed away from under the pitching by the action of the waves when receding, an action which would soon make itself felt in the case of a bank composed of light sand and gravel, if reached by the waves.

As a substitute for pitching, concrete slabs about 4 feet square and 5 inches thick, laid to break joint and bedded on gravel, are found to give good results, and are economical; they also look very presentable.

A layer of quarry rubbish is often laid underneath the pitching.

The height of the top of the bank above top water level depends upon the height to which the waves are likely to rise, a height which varies in proportion to the fetch, or length of exposed water above the dam, and which rarely exceeds 4 feet.

In works of considerable size the distance from normal top water level to the top of the dam may be as much as 7 feet, and is rarely less than 4 feet in the smallest examples. As in valley reservoirs, the normal top water is the level of the bye-wash, and in floods the water level may rise from 1 foot 6 inches to 2 feet above the bye-wash; the greatest variation between extreme top water level and the top of the bank is in all but very unusual cases 5 feet. It must also be remembered that a certain thickness of earth is necessary to cover the top of the puddle wall, and to prevent the heat of the sun from drying out the moisture in the clay and causing it to crack. About 3 feet is sufficient thickness to allow for this purpose.

The form of the inner slope of an earthen dam is not invariably that of a plain surface. In some cases the bank from the normal top water level upwards is built vertical, faced with brick or concrete, the planes being joined by a curve. This arrangement may be adopted for either of two reasons, or for both combined. The vertical wall is intended to check the rush of waves up the slope and prevent their rising to such a height as to pass over the top of the bank and possibly cut into it and thereby endanger its stability; or it may be built where a great top width is compulsorily adopted to reduce the quantity of material which forms the bank or dam. That the vertical wall does prevent the waves from reaching so high as they would do on a sloping bank, or that a dwarf wall placed on the top of the bank is a safeguard in this respect, is doubtful, as with the

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sloping face the water will be sprayed, by the force of the wind which must accompany the wave action, on to the bank, with little if any injurious effect, while if a vertical face should intervene the water will be thrown upwards, and being caught by the wind, fall on to the outer slope of the bank, and may injure it.

Concrete Walls.—In a few instances in this country, and more frequently in America, a wall of concrete or rubble has been employed, and sometimes with success, in lieu of puddle, to form the heart of an earthen embankment, being rendered watertight by a coating of cement mortar on the up-stream face.

There is evidence that many of the rubble centre walls in American earth dams leak to a serious extent, and where the whole wall both

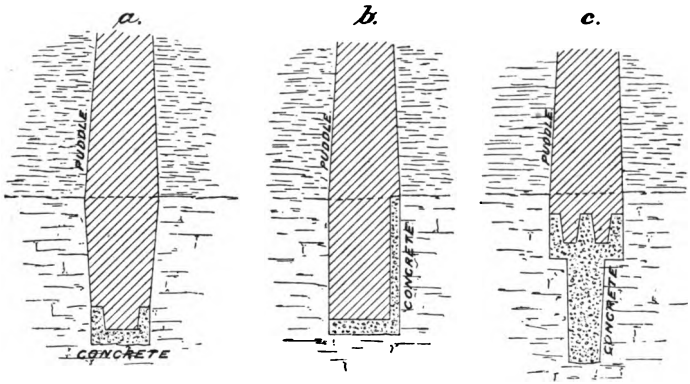

FIG. 24.

above and below ground is built as a monolith it is easy to understand how the wall may fail at or above ground level; while, if the masonry be divided at the ground level or the upper portion of the wall be made of puddle, the formation of a watertight joint between the two sections is a matter of difficulty.

Although the concrete wall system has not obtained favour in this country, an expedient which is frequently adopted and which has much to recommend it, is to form a concrete filling to the trench below the ground level. This concrete may take several forms. It may fill the trench entirely, taking the place of the puddle, or it may be formed into a lining entirely surrounding it, or may be used merely as an apron on the lower side of the clay. Some of these forms are shown in fig. 24.

When the trench is entirely filled with concrete, the top must be

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formed into tapering V-shaped grooves in order to effect a satisfactory junction between the concrete and the puddle above, so as to prevent the water from creeping through between the two materials.

It will be seen that when the grooves are formed as shown in fig. 24 (*c*), any subsidence of the puddle only effects greater compression of the material in the groove and consequently increased water-tightness.

Diverting Stream.—Before the embankment can be constructed some means must be provided by which the water of the stream to be impounded can be diverted, or at any rate carried over the site in such a way as not to interfere with the progress of the works.

One way of effecting this, more especially applicable in cases where the sides of the valley are not precipitous, is to divert the stream into a new channel to be excavated at a level which will, until it has passed the site of the dam, be at all points above the proposed top water level of the reservoir when finished.

Such a channel as this, if made in a permanent manner, will not only serve the purpose of keeping the site of the embankment dry during construction, but will be useful afterwards as a means of disposing at will of the first washings of the land at the commencement of a heavy flood, when the water may bring down with it a large quantity of deleterious matter in suspension, which it would be undesirable to pass into the reservoir.

Catch Pits.—At the head of the reservoir, at the point where the new channel leaves the original bed of the stream, a 'residuum lodge' or catch pit should be provided, through which the water passes on its way to the reservoir.

This catch pit serves the useful purpose of retaining any considerable detritus brought down by the stream, and consists broadly of a masonry tank of considerable area, sometimes in duplicate for convenience in working, provided with an outlet valve at the bottom, so that it may be flushed out into the bye-channel above mentioned. If the water should contain a large quantity of suspended matter, and the available fall should be sufficient, two or more of these catch pits may be arranged, one below another in a series of steps. Greater opportunity is thus allowed for settlement.

Leaping Weirs.—In this connection mention may be made of the so-called leaping weir, an appliance frequently used for causing turbid and impure flood waters to be automatically rejected.

The leaping weir is especially useful in such a situation as the

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catchwater, or collecting conduit, assumed to exist in our typical reservoir, shown in fig. 21.

It consists in its simplest form of a gap in the bed of a stream at right angles to the direction of flow, of such a width that the normal quantity of water passing such stream will fall through it into the catchwater channel crossing at right angles below, and will thus be conveyed to the reservoir. Should the stream, however, become unduly swollen with flood water, the velocity of the flow will be increased, and the water will leap over the opening without falling through, and will thus pass to waste down the original course of the stream.

In fig 25, A is the stream bed, and B the slot through which the normal flow passes to the collecting channel C. The dotted stream

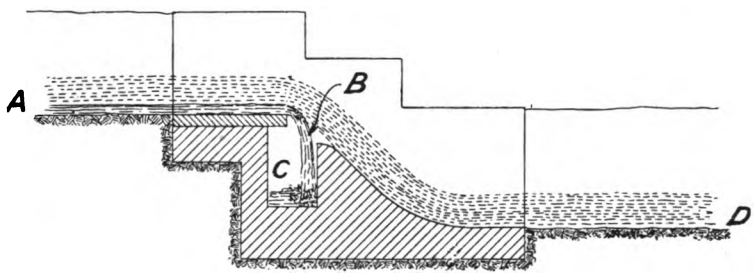

FIG. 25.

lines indicate the course of the water down the original stream-bed D, when the flow is above the normal.

In this simple form of the apparatus the width of the opening is fixed once for all by experiment, and cannot be altered to suit circumstances.

It may, however, be found that it is desirable to exclude water from the collecting channel on occasions when the flow in the stream is not so great as it would be in an actual flood, or it may be required to shut off the supply altogether for repairs or other reasons, in which case it is a convenience that the weir should be adjustable within certain limits, and should be capable of being closed entirely if required.

Such adjustable weirs are comparatively simple in construction, and will be found illustrated in different forms in the catalogues of various makers of waterworks appliances.

Tunnels or Culverts.—The other method of disposing of the

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waters of the stream during the construction of the dam is to construct a culvert or tunnel of masonry or concrete, or, if the flow be very small, a cast iron pipe may be used. This culvert, which must be large enough to permit of the passage of the heaviest flood likely to occur during the time when the reservoir is being made, although quite permissible, in fact usual, in a masonry dam, will be a source of weakness in an earthen bank, if it be constructed to pass through the puddle wall, and is liable to be fractured in consequence of the unequal settlement of the puddle, and the material composing the main body of the bank. It is needless to point out that such a breakage may be the cause of the destruction of the bank by reason of the leakage caused at the break, the immense damage ensuing lower down the valley being out of all proportion to the initial economy effected by this method of construction. This danger does not exist in the case of a masonry dam where the culvert forms merely a passage through the body of the work, suitably controlled by valves.

Culverts of this kind, where the bank is of earth, are now invariably superseded by tunnels driven through the solid ground, so as to avoid any interference with the bank.

Valve Towers.—When applied to an earthen embankment, the culvert is connected at the inner or up-stream toe of the bank to a 'valve tower' or 'upstand' (fig. 26), a tower, usually circular, of masonry, concrete, or, in some examples, of cast iron, which contains within it a vertical cast iron pipe, which is a continuation of the outlet main which passes through the culvert. At several different levels depending in their number on the depth of water to be impounded (usually one for every 15 feet or thereabouts), branch pipes are attached to the vertical main, and upon each branch a valve is inserted so that water may be delivered from any level desired, preferably from the surface, or as near thereto as possible, it being found that the water at the higher levels is much freer from suspended matter than that lower down.

Separate sluices may also be provided on the outside of the tower, one on each inlet pipe, for use in the event of the inner valve requiring repair.

In an arrangement of this kind, the interior of the tower, and of the culvert connecting with it, is quite dry, and the pipes are accessible through the whole length.

A separate main, controlled by a valve at the inner end, is

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provided for the purpose of regulating the amount of water to be passed out of the reservoir for compensation to the stream lower down the valley. This outlet is placed at the bottom of the reservoir, the quality of the water sent out not being of importance.

In some examples, the valve tower itself is made to act as a standpipe. In this arrangement, sluices are provided on the outside of the tower at different levels, but the water merely passes through them to gain access to the interior of the tower, whence it passes away either into sluices placed at different levels in the upstand pipe, or direct through a delivery main which connects with the base of the tower. The culvert is joined to the base of the tower, and is stopped at some point in its length, probably under the puddle wall, by a plug of concrete or brickwork, preferably slightly conical in form, this shape being adopted in order that the pressure of water due to the head in the tower may tend to drive the plug more tightly into its seat.

Puddle Collars.—The main which passes through the plug has cast upon it, or bolted round it, a projecting flange of considerable depth, known as a 'creeping flange' or 'puddle plate,' to prevent leakage along the pipe.

A similar ring of masonry or concrete may advantageously be formed round the outside of the culvert itself on each side of the puddle wall, the inner surfaces of the pair of rings being inclined so as to form a flat bottomed V-shaped space, into which the puddle tends to be compressed under pressure, in the same manner as in the case of the concrete toe previously described.

Comparison of Outlet Arrangements.—A drawback to the use of towers with no standpipe in the interior is that no duplicate sluices exist on the outside, so that any valve which may be in need of repairs can only be attended to by running the water off till the surface falls below that particular valve, a costly and, in many cases, impossible proceeding.

On the other hand, a wet tower forms a convenient means of screening impurities from the delivery main by the use of movable screens of copper-wire mesh of a fineness suited to the quality of the water, arranged in duplicate grooves extending across the diameter of the tower so as to divide it into two compartments, the one containing the sluice openings from the reservoir, and the other the entrance to the delivery main.

Duplicate grooves should be provided, in order that a clean set of

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screens may be lowered into either set of grooves before the dirty set is removed from the other to be washed. Needless to remark, the washing water must be conducted to the waste channel, and not returned to the reservoir.

The safest plan of all, but the most costly, is to conduct the discharge through a tunnel, round the end of the embankment or through the side of the valley into another watershed, as may be found most convenient. By this means all risk of settlement, or injury to the culvert from pressure by the bank above, is eliminated. If the outlet is led to the valley below the reservoir, the water for compensation may be taken in a main alongside the delivery.

The entrance to the tunnel is commanded by a valve-tower in the same manner as described for the culvert passing through the bank.

Syphon Outlets.—In cases where the reservoir is not above, say, 25 feet deep, a means of drawing off the contents is afforded by a syphon pipe, carried up the inner slopes, over the top of the bank and down the outer slope, to a point lower than the bottom of the reservoir, by at least the head necessary to overcome the friction in the pipe necessary to the discharge of the required quantity of water.

Valves are placed on the pipe on the inner slope at different points, to enable the water to be drawn off at any level required; and if required, these valves may be covered by wire screens, running on guides, so as to be capable of easy inspection and cleaning.

Means must be provided for drawing off the air which will in course of time accumulate at the crown of the pipe, otherwise the flow will be throttled, and in time will cease altogether.

In order to start the syphon the valves at both ends are closed, and the pipe filled with water from the top. The valve at the top being closed and the others opened, the action will be started, and will continue, unless air is admitted or allowed to accumulate at the crown.

The pipe may be charged, but not so conveniently, by exhausting the air by means of an air-pump, the lowest inlet valve alone being open, until the pipe is completely filled.

Syphon outlets are not to be recommended, unless no other arrangement can be adopted. They require a great deal of attention, and are generally unsatisfactory.

Sections of a valve tower and a syphon draw-off will be found in figs. 26 and 27.

Overflow.—Every storage reservoir must be provided with some

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means by which surplus water may be harmlessly disposed of when the reservoir is already full, a condition of things which frequently

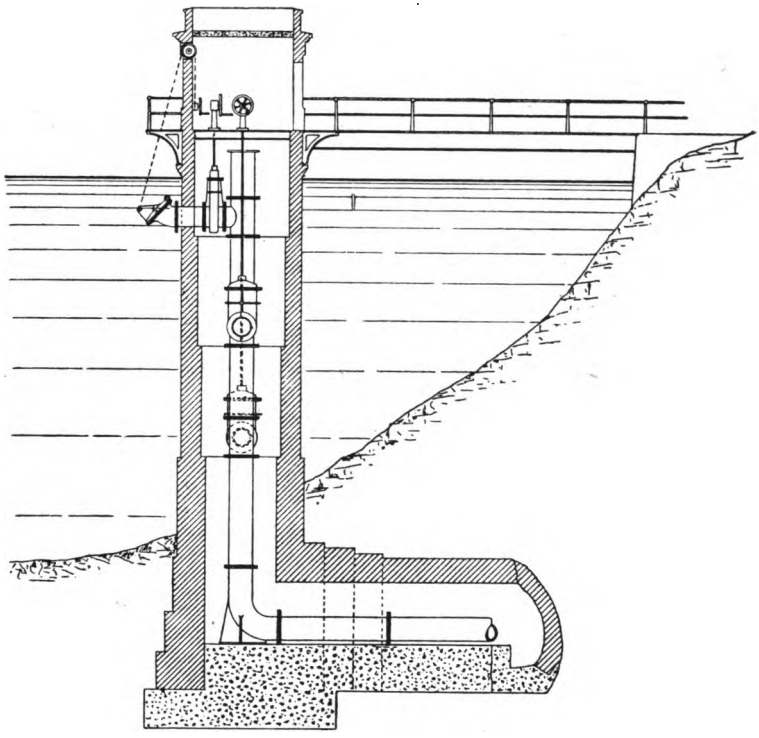

FIG. 26.

occurs during winter rains. It is evident that this excess of water cannot be allowed to run down the face of an earthen bank, or the bank would very soon cease to exist.

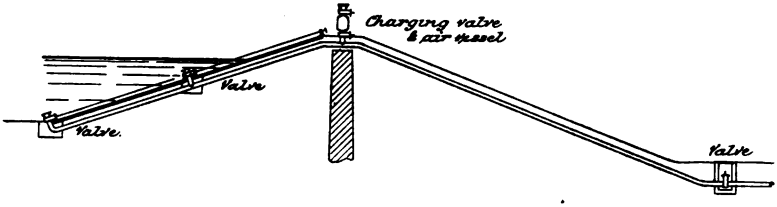

FIG. 27.

An overflow weir, or bye-pass, is therefore provided, having a flat sill of a length dependent upon the anticipated flow, and so proportioned that the maximum depth of water upon it will not exceed a

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certain figure, generally 2 feet. After passing over the sill with a free fall, the stream of water is conducted down the so-called 'waste-water course,' to join the original bed of the stream below the embankment. The sill of the weir is usually constructed of solid masonry, set in the strongest possible manner upon a firm foundation of concrete, the weir and waste-water course being both kept entirely at the side of the artificial embankment upon the solid undisturbed ground.

A concrete or masonry apron is laid on the inner side of the weir, and a bridge is usually necessary to give access to the embankment.

The waste-water channel should be so constructed as to ensure that the water in its course shall nowhere attain a velocity greater than that acquired in its fall over the weir. This object is attained by making a series of steps in the form of pools, thus providing a succession of weirs, each weir having beneath it a water cushion. The whole series of weirs should be built in the best masonry on a substantial concrete foundation.

The length of the overflow weir may be ascertained, if gaugings of the stream have been kept over a long period, from the formula $l = \frac{Q}{8}$, in which l = the length of the weir in feet, and Q = the maximum flood discharge in cubic feet per second.

This corresponds to a maximum height above the sill of about 1 foot 10 inches, the formula of discharge being the same as that given on page 56 for rectangular notches.

If no records of flow exist the length must be approximately arrived at by reference to the known rainfall. A formula which may be used is:—$l = 2R_m A^{\frac{1}{2}}$, in which R_m is the observed average rainfall, and A the number of acres in the catchment basin.

This formula is based on experimental observations of flow in England. The *square root* of the catchment area enters into it, in consequence of the fact that the larger the area the longer the time taken for the rain to reach the reservoir. In countries where very heavy rains are likely to continue for long periods, it is better to find the length required by equating the discharge as calculated from the formula on p. 56, to the effective maximum rainfall on the whole area.

An approximate rule, given as the minimum requirement by the Local Government Board, is that 3 feet in length of weir should be allowed for every 100 acres of gathering ground.

None of these expressions take into account the effect of an

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increase in the size of the reservoir in storing up and equalizing the flow due to a sudden rainfall.

Compensation Water.—The water for compensation purposes having been brought through, over, or round the embankment, is delivered usually into a gauge basin, arranged with suitable division walls, in such a manner that it shall fall over a weir without any velocity other than that due to the contraction over the lip.

The weir is fitted with an accurately levelled and sharp edge of cast iron, this edge being as thin as is consistent with durability, the level of the water in the basin being level with the top of the weir plate when the compensation required to be given is afforded.

The arrangements for gauging the compensation water are, as a rule, subject to the inspection of the riparian or mill owners below the reservoir who may be interested.

The manner in which the water is to be delivered is commonly specified in the Act of Parliament under which the works are constructed, and varies in different cases.

On rivers in the northern counties which are chiefly of use as sources of water power, it has been usual to allow to the stream a quantity equivalent to one-third of the available yield of the gathering ground, and this amount being required for power purposes, was specified to be all delivered between 6 a.m. and 6 p.m. every week day, with perhaps a small flow during the night, and on Sundays, to satisfy sanitary requirements.

Recently, however, the local authorities and river conservators have taken steps to secure that the flow shall be constant and evenly spread over the twenty-four hours, as being more generally to the interest of the population at large, and particularly to those at some distance below the reservoir, leaving the mill owners to take steps to head back the water for their own requirements at their own immediate expense.

In districts where the water is required for fishing purposes or to preserve the amenities of the river rather than as a source of power, it is considered sufficient to provide one-fifth of the available yield, or sometimes as little as one-sixth or one-seventh, but no fixed rule can be given, and compensation is frequently paid in money instead of water.

If the whole of the water available be not impounded, compensation is usually given only on that which is actually taken.

As an instance of an earthen bank, may be cited the Staines

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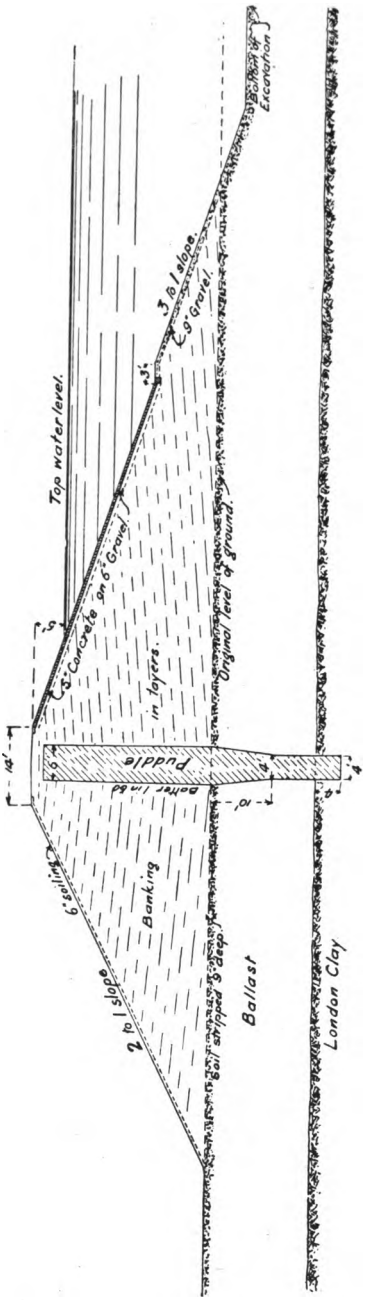
Top water level.
3 to 1 slope
6" Gravel.
6" Concrete on 6" Gravel
in layers.
Original level of ground.
Excavation
Puddle
Outer line of
Banking
6" soil
2 to 1 slope
Ballast
London Clay

FIG. 28.

reservoirs, recently completed by a Joint Committee of the Grand Junction, New River and West Middlesex Water Companies.

In this case the reservoirs are entirely artificial, the bank all round and the dividing bank in the centre being formed of material excavated within the area. On account of a slight difference of level in the ground, the bank varies in height somewhat at different points, but on the average is as shown in section in fig. 28.

The bottom is formed by the London clay, which is here covered by a stratum of gravel of varying thickness, through which the puddle wall is sunk until a good junction with the clay is secured; thus the clay for puddle, gravel for banking, and concrete, are all obtained from the interior of the reservoirs, most of the space left being available for the storage of water.

The embankments are 14 feet wide at the top, with an outside slope of 2 horizontal to 1 vertical, and an inside slope of 3 to 1, except the bank dividing the two reservoirs, which is, of course, 3 to 1 on both sides.

The puddle wall is 6 feet thick at the top with a batter of 1 in 60 on each side to the level of the ground; from the surface of the ground there is a reverse batter as shown. The puddle wall is continued downwards into the blue clay to a depth of 4 feet.

Before the embankments were commenced the bases were stripped of all surface soil and vegetable matter, which was preserved for after use in covering the outside slope of the bank.

All field drains below the embankments were taken up and their tracks filled with puddle.

The puddle trench was sunk on the centre line of the top of the embankments to a depth of about 4 feet into the London clay, and the refilling of the trench was commenced at the lowest point and carried on continuously right and left so as to have as few joins as possible.

The puddle was made from clay from which all stones of more than 3 inches in their greatest diameter were removed. It was then allowed to sour for at least twenty-four hours and worked up in a pug-mill. The layers in the trench were 3 inches thick, well cut and trodden in, so that each layer was thoroughly incorporated with that below it.

Care was taken that the wall was always at least 6 inches but never more than 12 inches above the surrounding banking, and that all transverse planks, rances, or other timbers were removed from

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it and from the trench. No puddling was allowed during frosty weather, nor were any frozen lumps permitted in the puddle.

The embankments were formed in layers not thicker than 9 inches and having a slope of 1 in 12 towards the puddle wall, moistened with water and well beaten or rolled. All stones exceeding 3 inches in diameter were placed at least 4 inches apart and thoroughly blended with fine material. No vegetable matter was permitted in the banks except the surface soiling.

The inner face of the embankments is protected to a depth of 15 feet below top water level, by concrete slabs made, *in situ*, of 4 to 1 concrete, 4 feet square and 5 inches thick, worked to a good face and resting on 6 inches of gravel, every alternate slab being left out till subsidence had ceased, when those in position were packed up to the true line of slope and the work completed. At the foot of the concrete facing is a berm or step 3 feet wide, and below that line the embankments are covered with a layer of gravel 9 inches thick.

A roadway is formed along the top of the banks, with an edging of turf.

The whole of the outer slopes of the embankments are covered with soil 6 inches thick resting on 3 inches of gravel, and sown with clean rye grass and white clover seeds.

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CHAPTER V

MASONRY DAMS

In America, in India, as well as in other countries, dams built entirely of masonry have been extensively used, and several notable examples have been constructed in this country.

Masonry dams are cheaper than earthen embankments, where the height is great, and in places where suitable material for a puddle wall cannot readily be obtained, and where the foundation is solid enough to bear the extra pressure per unit, due to the smaller thickness of the masonry dam.

Stability.—The remarks as to the selection of site, etc., apply to masonry as well as earthwork dams, and it only remains to consider the dimensions which the former must have to ensure stability and the strength of the foundations.

To understand the necessary conditions of stability, we will consider the forces on the dam, how they act, and the effect of each.

Taking unit length of any dam, A B C D, the forces are: (1) the weight, W, of the dam itself acting vertically downwards through its centre of gravity, G; (2) the force, F, due to the pressure of the water on the inner face.

The magnitude and direction of this force F will depend upon the depth of the water, and upon the shape of the inner face. If the latter be not vertical the force is compounded of that, F_1, on a corresponding vertical wall E C, together with the weight, F_2, of the intermediate wedge of water B E C. This last will act vertically downwards through the centre of gravity of the wedge. The force F_1 will act horizontally at a point one-third h from the bottom, h being the height of the water surface above the base C. The magnitude of F_1 will be $\frac{1}{2}wh^2$, where w is the weight of unit volume of the water.

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Now the resultant, R, of F and W will give the total force on the base of the dam.

If the dam could be regarded as one rigid, incompressible, and unbreakable whole, it would only be necessary to consider this one force R, and the only condition for stability of the dam would be that the line of action of R should fall inside the base of the dam.

But as a matter of fact the dam cannot be looked upon as a rigid whole, and it is necessary to consider how the force R is distributed over the different parts of the base.

It is evident that, with forces acting as shown in fig. 29, the dam, if it failed by overturning, would turn about the point D.

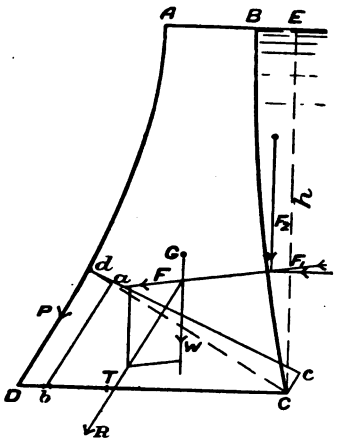

FIG. 29.

It is usual to assume that the pressure on the base, then, will be greatest at the end D, and diminish gradually and uniformly towards the end C, the word *pressure* being used to denote the *intensity* of the force at each point, or the force per unit of area, such area being perpendicular to the direction of the force.

If, then, lines *Dd* and *Cc* be drawn at D and C to represent the pressures at these points in magnitude and direction, they will be parallel to R, and if *dc* be joined, and any very small strip *b* of the base be taken, the line *ab* will represent the pressure at *b*, and if this be multiplied by the area of the strip in a direction perpendicular to the pressure, the result will give the force on that strip.

It follows from this: (1) that the total force R is represented by the area of the figure *d*DC*c*, being made up of the forces on the

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several strips; (2) that the force R must act through the centre of gravity of the figure dDCc.

Now, it is evident that the less masonry there is in the dam the greater will be the force F, in comparison with W, and the more nearly will R approach to D; also the less will be the cost of the dam.

Hence the dam will be cheapest when R is as close as possible to D. This is usually taken as being when the pressure at C is reduced to zero. If the breadth of the dam be still further diminished below that point, the pressure at C becomes negative; that is, the masonry there will be in a state of tension, and it is, in general, considered inadvisable to design masonry structures so that they will be anywhere in tension, if it can be avoided.

Suppose, then, that the pressure at C is zero, and join dC. We thus get a triangle dDC, and R, for the most economical safe dam, will act through its centre of gravity, or one third the breadth of the base from D.

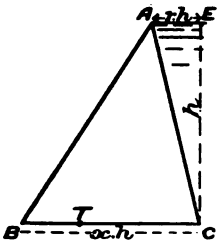

FIG. 30.

The condition for stability against overturning, then, is that the resultant force R must act within the centre third of the base, and we get the most economical dam by arranging that when the dam is full the sum of the moments of the forces W, F_1, and F_2 about the point T will be zero, when DT $= \frac{1}{3}$DC.

Result for Triangular Dam.—Applying this to a triangular dam, if we put

d = specific gravity of masonry,
w = weight of unit volume of water,
h = height,
xh = base = CB.
rh = inner slope = AE, and suppose the reservoir

full we get $F_1 = \frac{wh^2}{2}$,

distance above base $= \frac{1}{3}h$.

$\therefore$ moment $= \frac{1}{6}wh^3$ . . . . . (1)

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And moment of weight of wedge of water about T

$$=\tfrac{1}{6}wh^3r\,(2x-r) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

and moment of weight of wall $=\tfrac{1}{6}wh^3xd(x-r)$.

Whence, equating moments and simplifying,

$$1=x^2d-xr(d-2)-r^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If this be differentiated with regard to r, we find that x is at a minimum when $r=\dfrac{2-d}{\sqrt{d^2+4}}$, and that the corresponding or lowest value of x is $x=\dfrac{2}{\sqrt{d^2+4}}$.

Thus with such a dam, say 40 feet high, if the density of the masonry be 1·8 times that of water, the most economical base to give it would be $\dfrac{2}{\sqrt{1{\cdot}8^2+4}}\times 40=29$ feet 9 inches nearly, and the batter of the inner face would be $40\times\dfrac{{\cdot}2}{\sqrt{1{\cdot}8^2+4}}=3$ feet nearly.

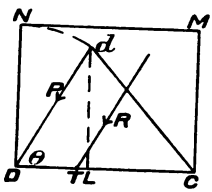

FIG. 31.

If the density of the masonry be greater than 2, it is evident that the value of r will become negative. That is, our solution would give us an overhanging wall.

This would be the most economical, if the reservoir were always full to the top, but it would be unstable when the reservoir was empty, and it would be difficult to build. Hence, in such a case r would be put $=0$, and the inner face made vertical. To find the proper value for x, we must then substitute $0=r$ in equation No. (3).

Whence $1=x^2d$,

$$\text{or } x=\frac{1}{\sqrt{d}}=\frac{\sqrt{d}}{d}.$$

Resistance to Crushing.—A dam thus designed would be quite safe against overturning. We will now consider whether it is likely to give by crushing, or by one part sliding upon the other.

We have already seen that if R be the total force on unit length of the base it will be unevenly distributed, and will produce, when

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the dam is well designed, a maximum pressure, P, at D, and no pressure at C, when the reservoir is full.

The condition for stability against crushing, then, is that the masonry must be capable of bearing the maximum pressure P. Thus make DN = P. Then the base must be capable of bearing everywhere a pressure represented by D N, and therefore a total force, *per unit length* of the dam, represented by the area of the rectangle D N M C.

Now $$dD = dL \operatorname{cosec} dDC,$$
and as already stated (p. 89),

$$R = \text{area of } DCd,$$
$$= \tfrac{1}{2} DC.dL.$$
$$\therefore\ dL = \frac{2R}{DC}$$
$$\therefore\ dD = \frac{2R \operatorname{cosec} dDC}{D\,C} \quad . \quad . \quad . \quad . \quad (4)$$

But dD must not exceed the safe pressure on the masonry which is known for different kinds of masonry.

R can be found for any given dam, being the resultant of the weight of the wall and the force due to the fluid pressure.

The angle dDC can also be found, for the vertical forces are F_1 and W (fig. 29), both of which are known, and the horizontal force is F_2, which is also known.

Hence by the principle of the composition of forces,

$$\tan dDC = \frac{F_1 + W}{F_2} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

We can proceed, then, in two ways, viz., either by taking the base D C found to be necessary to prevent overturning, and working out the right hand side of equation No. (4). The resulting value of dD should not exceed the safe value for the masonry.

Or we may put in this safe value for dD, and calculate the least value of D C from equation (4). If this be less than that required for overturning, the base must be widened accordingly.

Resistance to Sliding.—The condition for safety against sliding is that the angle dDC should not be less than the complement of the limiting angle of friction for the material. A wall is not at all likely to give in this way if the foundations are built with care, and the masonry is properly bonded.

Application to Triangular Dam. — These results can be readily applied to a dam such as that shown in fig. 30.

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For $W = \frac{1}{2}xh^2dw$, and $F_1 = \frac{1}{2}rh^2w$,

$$\therefore \text{ vert. forces} = \frac{wh^2}{2}(xd + r),$$

and
$$F_2 = \frac{wh^2}{2}.$$

Put angle $dDC = \theta$,

$$\therefore \tan\theta = xd + r = \frac{d+2}{\sqrt{d^2+4}},$$

substituting the values of x and r (p. 90).

$$\therefore \operatorname{cosec}^2\theta = 1 + \cot^2\theta = \frac{2(d^2+d+4)}{(d+2)^2}.$$

Now
$$dD = \frac{2R \operatorname{cosec}\theta}{DC} \ldots \text{ eqn. (4)},$$

$$= \frac{2(F_1 + W)\operatorname{cosec}^2\theta}{xh},$$

$$= \frac{2\frac{wh^2}{2}(xd+r)\frac{2(d^2+d+4)}{(d+2)^2}}{xh},$$

$$= wh\frac{d^2+d+4}{d+2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Thus if $d = 2$, the maximum stress on the material, supposing the height to be 40 feet, will be

$$62{\cdot}5 \times 40 \times \frac{10}{4} = 6250 \text{ lbs. per square foot.}$$

If 6 tons per square foot be the maximum safe pressure, we can find the greatest height to which this section could be carried, thus—

$$6 \times 2240 = 62{\cdot}5 \times h \times \frac{10}{4},$$

$$\therefore h = \frac{6 \times 2240 \times 4}{62{\cdot}5 \times 10} = 86 \text{ feet nearly.}$$

Tan θ will be $\frac{4}{\sqrt{8}} = \sqrt{2}$, $\therefore \theta = 54^\circ\ 44'$,

which is greater than the complement of the limiting angle of friction.

Objections to Triangular Section.—The triangular section is not, however, suitable for dams. One reason is that objects such as floating logs may find their way into the reservoir, and might possibly injure the thin edge at the top, and the same result might be produced by the pressure of ice. Moreover, a path along the top of a dam is always useful and often necessary, hence there is always a certain width allowed at the top.

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If any other section be taken we can investigate it in the same way, but in most cases, especially where the dam is a high one, the equations which result are so long and cumbersome as to be practically useless.

Hence there has been considerable diversity in the shapes that have been given to such dams.

Trapezoidal Dam.—We will first briefly investigate a low trapezoidal dam and a buttressed dam, and then give some standard sections.

Let t (fig. 32) = breadth at top, s = specific gravity of masonry, the other letters being used in the same sense as before.

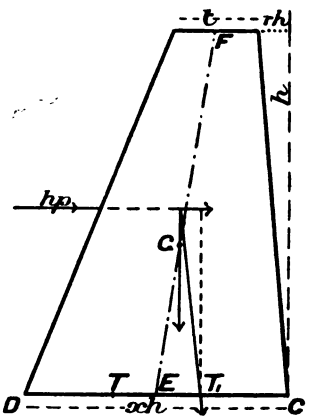

FIG. 32.

We will first take the dam empty, and find the relation between x, r, and t when the resultant passes through the point T_1, such that $T_1C = \frac{1}{3}DC$; taking into account a horizontal wind pressure, p, on the outer face.

The centre of gravity of the dam is on the line E F, and divides that line at G such that

$$EG = \frac{EF}{3}\left(\frac{xh+2t}{xh+t}\right),$$

$\therefore$ horizontal distance from $E = \left(\frac{xh}{2} - \frac{t}{2} - rh\right) \cdot \frac{xh+2t}{3(xh+t)} = y$, say.

$\therefore$ " " " $T_1 = \frac{xh}{6} - y,$

$$= \frac{rh(hx+2t)+t^2}{3(hx+t)}.$$

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And weight $= \frac{xh+t}{2} . hsw$,

$$\therefore \text{ moment } = \frac{hsw}{6}(h^2xr + 2rht + t^2).$$

Moment of wind pressure $= \frac{1}{2}h^2p$,

$$\therefore hp = \frac{sw}{3}(h^2xr + 2rht + t^2);$$

whence
$$r = \frac{3hp - swt^2}{sw(h^2x + 2ht)}, \quad . \quad . \quad . \quad . \quad . \quad (1)$$

This gives the required relation between x, r, and t for the empty dam. If the inner face be vertical, $r = 0$, and we get $3hp - swt^2 = 0$,

$$\text{or } t = \sqrt{\frac{3hp}{sw}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

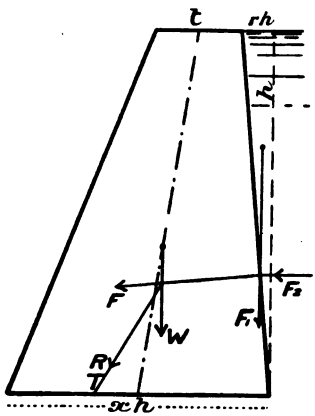

FIG. 33.

The following points should also be noted :—

(1) If t be made *greater* than this, r would be negative if the wall is to be everywhere just in compression, with the reservoir empty and the wind blowing up-stream.

(2) If t be made greater, but r kept at zero (*i.e.* with the inner face vertical), there will be a balance in favour of stability over and above the necessary amount.

(3) If t be made *less*, r being zero, the wall will be partly in tension with an empty reservoir and full wind pressure. In other words, when r is zero, the *least* value of t is $\sqrt{\frac{3hp}{sw}}$.

We will now consider the reservoir full. It is unnecessary here to consider wind pressure, because it is less than the water

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pressure on the inner face, and both cannot act, inasmuch as the water prevents the wind from blowing on the inner face (fig. 33). Wind pressure on the outer face would simply increase the stability, but as it is not always acting, we cannot rely on it.

Proceeding as before, but taking moments about T, we find,

$$\text{Moment of weight of wall} = \frac{hsw}{6}\{h(x-r)(xh+t)-t(rh+t)\}.$$

$$\text{,, ,, } F_1 \qquad = \frac{h^3wr}{6}(2x-r).$$

$$\text{,, ,, } F_2 \qquad = \frac{wh^3}{6}.$$

$\therefore$ equating moments and simplifying,

$$h^2 = h^2r(2x-r)+s\{h(x-r)(xh+t)-t(rh+t)\} \quad . \quad . \quad . \quad (3)$$

Differentiating to find the minimum value of x,

$$0 = h^2(2r\frac{dx}{dr}+2x-2r)+s(2h^2x\frac{dx}{dr}+ht\frac{dx}{dr}-h^2x-h^2r\frac{dx}{dr}-2th),$$

whence $\frac{dx}{dr} = \frac{2xh-2rh-s(xh+2t)}{2h(r+xs)+s(t-rh)}$.

$\therefore$ when $\frac{dx}{dr}=0$, or x is at its least value, we have

$$2xh-2rh-s(xh+2t)=0,$$

or

$$2rh = xh(2-s)-2st.$$

Now s will not, in general, be less than 2, and it is evident that if it be not less than 2, r will be negative for the minimum value of x.

Rejecting negative values of r, which would give overhanging walls, we get the best result (*i.e.* the nearest to the minimum value), by putting $r=0$. Substituting in equation No. (3), we get

$$h^2 = s(h^2x^2+hxt-t^2),$$

or

$$x^2sh^2+x.hst-(t^2s+h^2)=0,$$

$$\text{whence } x = \frac{\sqrt{h^2s^2t^2+4sh^2(t^2s+h^2)}-hst}{2sh^2}$$

$$= \frac{\sqrt{s^2t^2+4s(t^2s+h^2)}-st}{2sh} \quad . \quad . \quad . \quad . \quad (4)$$

Thus if any given value be fixed for t, we can find x. Now we get

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the least masonry in the wall when $xh+t$ is as little as possible, and from equation (4),

$$xh+t=\frac{\sqrt{s^2t^2+4s(t^2s+h^2)}-st+2st}{2s}$$

$$=\frac{\sqrt{s^2t^2+4s(t^2s+h^2)}+st.}{2s.}$$

Obviously this increases with t, and hence we should take the least value for t.

But we have already seen, that for stability when the dam is empty, this is $\sqrt{\frac{3hp}{ws}}$.

∴ substituting this value for t in equation (4), we get the best value of x.

Equation (4) then reduces to

$$x=\frac{\sqrt{15p+4hw}-\sqrt{3p}}{2\sqrt{hws}} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Thus suppose $h=35$ feet,
$s=2{\cdot}4$ „
$p=40$ lbs. per square feet.

Then $$t=\sqrt{\frac{3hp}{ws}}$$

$$=\sqrt{\frac{3\times35\times40}{62{\cdot}5\times2{\cdot}4}}=\sqrt{28}=5{\cdot}3 \text{ feet,}$$

and from equation (5),

$$x=\frac{\sqrt{15\times40+4\times35\times62{\cdot}5}-\sqrt{120}}{2\sqrt{35\times62{\cdot}5\times2{\cdot}4}}={\cdot}6 \text{ nearly,}$$

whence breadth at base = 21 feet nearly. So that the dimensions of the dam at the deepest part, down to ground level, will be—

Top thickness, say . .	5 feet	4 inches.
Bottom thickness, . .	21 „	0 „
Height	35 „	inner face vertical.

If the same thickness at top is retained all the way, and the same batter be given to the outer face, there will be an excess of stability in the lower parts. If this is not desired, the values of t and x can be calculated for several different heights.

Assuming a horizontal wind pressure of 40 lbs. per square foot,

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Table I. gives the values of t, and Table II. the values of xh, for several different values of s and h, all the dimensions being in feet.

Table I.

Top widths of trapezoidal dams, with inner face vertical, calculated from the formula $t=\sqrt{\frac{3hp}{ws}}$ ($p=40$).

Values of s.	Values of h (feet).									
	10	15	20	25	30	35	40	45	50	60
2·0	3·1	3·8	4·4	4·9	5·4	5·8	6·2	6·6	6·9	7·6
2·1	3·0	3·7	4·3	4·8	5·2	5·7	6·0	6·4	6·8	7·4
2·2	3·0	3·6	4·2	4·7	5·1	5·5	5·9	6·3	6·6	7·2
2·3	2·9	3·5	4·1	4·6	5·0	5·4	5·8	6·1	6·5	7·1
2·4	2.8	3·5	4·0	4·5	4·9	5·3	5·7	6·0	6·3	6·9
2·5	2·8	3·4	3·9	4·4	4·8	5·2	5·5	5·9	6·2	6·8
2·6	2·7	3·3	3·8	4·3	4·7	5·1	5·4	5·8	6·1	6·7

Table II.

The thicknesses at ground level of trapezoidal dams, of different heights.

Values of s.	Values of h (feet).									
	10	15	20	25	30	35	40	45	50	60
2·0	6·3	9·5	12·8	16·1	19·4	22·7	26·0	29·3	32·7	39·5
2·1	6·2	9·3	12·5	15·7	18·9	22·2	25·4	28·6	31·9	38·5
2·2	6·0	9·1	12·2	15·3	18·5	21·7	24·8	27·9	31·2	37·6
2·3	5·9	8·9	11·9	15·0	18·1	21·2	24·3	27·3	30·5	36·8
2·4	5·8	8·7	11·7	14·7	17·7	20·8	23·8	26·8	29·9	36·0
2·5	5·7	8·5	11·4	14·4	17·3	20·3	23·3	26·2	29·3	35·3
2·6	5·6	8·4	11·2	14·1	17·0	19·9	22·9	25·7	28·7	34·6

If it be desired to make t less than the values here given, the dam will be partly in tension against the full wind pressure if the inner face be vertical and the reservoir empty; but if t be made *greater*, there will be an excess of stability with the empty reservoir. In either case, the least value of x may be found by substituting the selected value for t in equation No. 4.

Pressure on Base.—We have seen that with such a dam, when the reservoir is full, the weight, W, of the wall is $\frac{xh+t}{2} \cdot hsw$, where $s=$ specific gravity of masonry, and $w=$ weight of unit volume of water. Also horizontal water pressure $=\frac{wh^2}{2}$.

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∴ If R be the resultant force on base, its magnitude is given by the equation

$$R^2 = \left(\frac{hsw}{2}\right)^2 (xh+t)^2 + \left(\frac{wh^2}{2}\right)^2$$

$$= \left(\frac{wh}{2}\right)^2 \{s^2(xh+t)^2 + h^2\} \quad . \quad . \quad . \quad . \quad (6)$$

And the resultant makes an angle θ with the horizon, such that

$$\tan\theta = \frac{\frac{hsw}{2}(xh+t)}{\frac{wh^2}{2}}$$

$$= \frac{s(xh+t)}{h}.$$

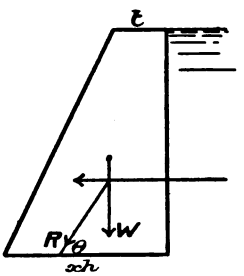

FIG. 34.

Now, we have seen (p. 91) that if P be the maximum pressure on the material—*i.e.* the pressure at the outer edge or *toe* of the dam) —then $\frac{1}{2}P.xh.\sin\theta = R$.

$$\text{Whence } P = \frac{2R}{xh\sin\theta}.$$

$$\text{But } R = \frac{W}{\sin\theta} \quad \therefore P = \frac{2\,W}{xh\sin^2\theta}$$

$$= \frac{(xh+t)h.s.w.}{xh\frac{\tan^2\theta}{1+\tan^2\theta}}$$

$$= \frac{(xh+t).sw.\{h^2+s^2(xh+t)^2\}}{xs^2(xh+t)^2}$$

$$= \frac{w\{h^2+s^2(xh+t)^2\}}{xs(xh+t)} \quad . \quad . \quad (7)$$

This should not exceed the safe pressure on the material. We will test it for a dam 60 feet high, taking $s = 2{\cdot}6$.

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From the tables, $t = 6{\cdot}7$, say 7 feet.

$$xh = 34{\cdot}6 \text{ feet, whence } x = \frac{34{\cdot}6}{60} = {\cdot}58.$$

$$\text{Substituting } P = \frac{62{\cdot}5\{3600 + 2{\cdot}6^2(34{\cdot}6+7)^2\}}{{\cdot}58 \times 2{\cdot}6(34{\cdot}6+7)},$$

$= 15{,}240$ lbs. per square foot, or 6·8 tons per square foot, nearly.

This pressure may be considered somewhat high. The high specific gravity (2·6) would, however, point to first-class masonry, which would probably stand a great deal more with perfect safety.

Table No. 3 gives the maximum pressures in tons per square foot at the base of such dams, as calculated from equation No. (7), using the dimensions in Tables I. and II.

TABLE III.

Specific Gravity.	Values of h in Feet.									
	10	15	20	25	30	35	40	45	50	60
2·0	1·07	1·55	2·01	2·47	2·92	3·38	3·83	4·29	4·74	5·61
2·1	1·12	1·60	2·08	2·56	3·03	3·50	3·96	4·44	4·90	5·82
2·2	1·16	1·66	2·16	2·66	3·15	3·64	4·11	4·60	5·06	6·02
2·3	1·19	1·71	2·24	2·73	3·24	3·74	4·24	4·74	5·24	6·22
2·4	1·23	1·76	2·30	2·83	3·35	3·86	4·38	4·90	5·40	6·42
2·5	1·27	1.82	2·38	2·93	3·46	3·99	4·51	5·06	5·57	6·62
2·6	1·31	1·88	2·46	3·04	3·57	4·11	4·65	5·21	5·74	6·82

It will be noticed that the pressure in the majority of these dams is very much below the limit of 6 tons per square foot. Hence it is probably in many cases cheaper—at any rate in places where the cost of the material is great—to build buttressed dams, if the height is not great.

Before discussing such dams, it may be mentioned that in no case does a trapezoidal dam give the minimum amount of masonry required.

Given a certain thickness at the top, the first part of the wall will be vertical, followed by a curved section, for the greatest economy of material.

It is also to be borne in mind that at some parts of the dam top water level is below that of the crest of the dam. At the overflow

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weir the water level may be as much as 2 feet above the crest of the wall. Strictly speaking, in our calculations the conditions most unfavourable should be taken into account at the deepest part of the dam.

We have taken the reservoir as being full just to the crest of the dam. The more exact method of calculation would, in most cases, result in little difference, and it is hoped that the examples given will enable the student possessed of fair mathematical knowledge to calculate any similar case.

Foundations for Dam.—In the preceding observations the strength of the masonry to resist crushing has been taken as the governing element, and it will be so in cases where the dam is built on a base of rock of great density; but it is possible for a masonry dam to be built on a rock foundation the density of which is less than that

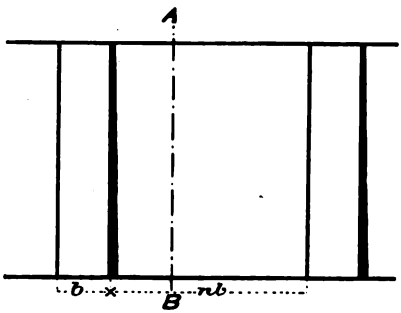

FIG. 35.

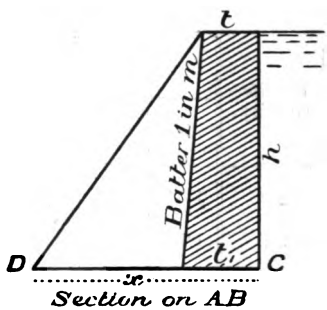

FIG. 36.

of the masonry, or it might be built on clay the safe pressure on which would not exceed from 3 to 3½ tons per square foot. In either of the above cases the area of the base of the dam must be extended accordingly; it will also be understood that the same care must be taken in rendering the attachment of a masonry dam to its base watertight, as with the puddle trench and wall of an earthen dam.

Buttressed Dams.—It has been shown that an economy in material can be effected by buttressing low dams. Whether the saving will be real or not will depend upon the special circumstances of each case, as the cost of construction per cubic yard of masonry will be increased by buttressing.

It is proposed to briefly discuss the mathematics of the buttressed dam.

Suppose the dimensions to be those shown on figs. 35–37, the inner face being vertical.

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Then if the pressure be zero at the inner edge C of the base, and increase uniformly to the toe D of the buttress, the stress diagram will be that shown in fig. 37, where the wedge C A D will represent the total force on the base of the buttress, and P the maximum pressure there. The volume of the wedge C B E F will represent the total force on the wall between two buttresses, and the line F G the maximum pressure there.

Thus for the pressure to be just zero at the inner edge, the resultant force on the whole $(b+n\,b)$ feet of the wall must act through the centre of gravity of the whole figure A D E B C.

Taking moments, it will be found that the distance of this point from B C is $\frac{2(x^3+nt_1^3)}{3(x^2+nt_1^2)}$. Whence this gives the distance from C

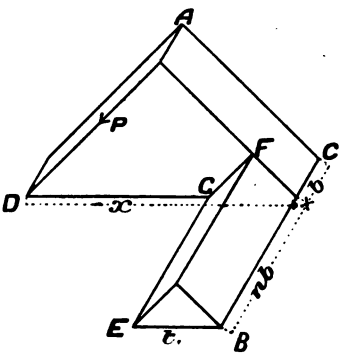

FIG. 37.

at which the resultant force must act, and therefore the point in the base about which the moments of the water pressure and the weight of the wall must be equal.

We now, therefore, take moments about that point, and equate. The following is the result after simplification :—

$$2s\,.\,\frac{x^3+nt_1^3}{x^2+nt_1^2}\{n(t_1+t)+x+t\}$$
$$=s\{n(t_1^2+tt_1+t^2)+x^2+tx+t^2\}+(n+1)h^2, \text{ or}$$
$$sx^4+sx^3\{2n(t_1+t)+t\}-x^2\{s\,nt_1(2t_1+t)+(n+1)(st^2+h^2)\}+x.sn\,t_1^2(2t_1-t)$$
$$=nt_1^2\{(n+1)(h^2+st^2-2st\,t_1)-nt_1\,s(t_1-t)\}.$$

The equation for x is thus a quartic and difficult of exact solution. But it can be pretty readily solved by trial with quite sufficient accuracy for practical purposes, selecting values for n, t, and t_1.

Worked Example.—Thus suppose $h=20$ feet, $s=2{\cdot}4$, $n=3$.

Take $t=2$ feet, $t_1=3$ feet, allowing a batter of 1 in 20.

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Then the equation reduces to—

$$2.4x^4 + 2{\cdot}4 \times 32x^3 - x^2\,(1600 + 2{\cdot}4 \times 88) + 2{\cdot}4x \times 108$$
$$= 27\,(1600 - 2{\cdot}4 \times 23),$$
$$\text{or } x^4 + 32x^3 - 754\tfrac{2}{3}x^2 + 108x = 17{,}385.$$

Try $x = 18$

$$\begin{aligned} x^4 &= 104{,}976 \\ 32x^3 &= 186{,}624 \\ 108x &= 1{,}944 \\ \hline & \ 293{,}444 \\ 754\tfrac{2}{3}x^2 &= 244{,}412 \\ \hline & \ 49{,}032, \text{ which is too great.} \end{aligned}$$

Try $x = 17{\cdot}0$, and write the equation—

$$x^2(x^2 + 32x - 754\tfrac{2}{3}) + 108x = 17{,}385$$
$$x^2 + 32x = 17 \times 49 = 833$$
$$754\tfrac{2}{3}$$
$$78\tfrac{1}{3}$$
$$x^2 \times 78\tfrac{1}{3} = 289 \times 78\tfrac{1}{3} = 22{,}639$$
$$108x = 1{,}836$$
$$24{,}475, \text{ still too great.}$$

The figures, however, show a drop of 25,000 (from 49,000 to 24,000) for 1 foot. A farther drop of 7000 being required (from 24,000 to 17,000), it is clear that 16·7 feet, or about 16 feet 9 inches, will be sufficiently exact for the value of x.

Any other case can be treated in the same way. The following table gives the approximate values of x, the values ascribed to the different symbols being stated in the table.

Values of h in feet	10	15	20	25	30
Values of n	3	2·5	2	1·5	1
,, b	1·5	2	2·5	3	4
,, nb	4·5	5	5	4·5	4
,, t	2	2·5	3	3·5	4
,, t_1	2·5	3·25	4	4·75	5·5
Values of s.	Values of x in feet.				
2·0.	8·4	12·4	16·3	19·8	22·8
2·2.	8·0	11·8	15·5	18·8	21·6
2·4.	7·7	11·2	14·7	17·9	20·6

It will be noticed that arbitrary values have been given to the

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letters n, t, and b (*vide* p. 100) in each case, and a batter of 1 in 20 assumed in finding t_1.

Tensile Stress.—It remains to test the pressure at the toes of the buttresses, and to find whether the intermediate parts of the wall will be safe.

It is clear that if the down-stream faces of these intermediate parts be arched in plan, taking the dimensions in the table as the least thicknesses, tension in the masonry can be avoided. But if they be built straight in plan from buttress to buttress, each intermediate part will, in effect, be a beam which is practically fixed at each end, and tension will be set up. We will treat them, however, as beams simply *supported* at each end, and calculate the tensile stress due to the bending moment of the water pressure.

Worked Example.—Take the dam 20 feet high. In the table a thickness of 4 feet at the bottom is ascribed to this, the space between the buttresses being 5 feet.

Considering the lowest foot, we have a beam 5 feet long, 1 foot broad, and 4 feet deep, loaded with the water pressure at a depth of 20 feet.

This pressure $= 20 \times 62{\cdot}5 = 1250$ lbs. per square foot.

$\therefore$ load per foot length $= 1250$ lbs.

$$\therefore \text{ bending moment at centre} = \frac{1250 \times (\text{length})^2}{8}$$

$$= \frac{1250}{8} \times 25$$

$$= 3910 \text{ feet lb. units nearly.}$$

Moment of resistance $= \dfrac{S \times (\text{depth})^2}{6}$, where S = maximum tensile or compressive stress.

$$= \frac{S \times 16}{6} = 2\tfrac{2}{3}S.$$

Equating to bending moment,

$$S = \frac{3910 \times 3}{8} = 1466 \text{ lbs. per square foot,}$$

$$= \tfrac{2}{3} \text{ ton per square foot nearly.}$$

Following the same system of calculation, we get the results given in table:—

Height.	10	15	20	25	30
Stress in (lbs. per sq. ft.).	1520	1660	1466	1050	744

In no case, therefore, is the tensile stress more than about ·75 ton

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per square foot, which any masonry that would be put into such a work could be safely relied upon to bear.

Compressive Stress.—To calculate the maximum compressive stress at the toe, consider the 20-foot dam, buttresses 2·5 feet thick and 5 feet apart, or 7·5 feet centre to centre (*vide* Table, p. 102), wall 3 feet thick at top, 4 feet at bottom, and take $S = 2{\cdot}4$, so that $x = 14{\cdot}7$ from the table.

Then taking P as the maximum pressure, the volume of the figure A B C D E (fig. 38) must represent the resultant force on the base (p. 101) or $R = (P \times 14{\cdot}7 \times 2{\cdot}5 + \frac{4P}{14{\cdot}7} \times 4 \times 5) \times \frac{\sin\theta}{2}$, where R is the resultant of the water pressure on 7·5 feet of the wall, and the weight of the same 7·5 feet.

The water pressure $20^2 \times \frac{1}{2} \times 7{\cdot}5 \times 62{\cdot}5 = 93{,}750$ lbs.

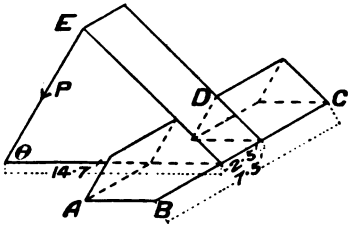

FIG. 38.

$$\text{Weight of wall} = \{3{\cdot}5 \times 20 \times 5 + 20\,(x+3) \times 2{\cdot}5 \times \tfrac{1}{2}\}\ 62{\cdot}5 \times s$$
$$= 118{,}875.$$

$$\therefore \tan\theta = \frac{118{,}875}{93{,}750}, \text{ whence } \theta = 51^\circ\ 37'.$$

$$\therefore R = 118{,}875 \operatorname{cosec}\theta.$$

Substituting this value for R in the above equation, and solving to find P, we obtain $P = 3{\cdot}36$ tons per square foot.

The following table shows the result of similar calculations for the other cases; the pressures being in tons per square foot:—

Values of *s*.	Values of *h*.				
	10	15	20	25	30
2	2·13	3·07	3·69	4·09	4·23
2·2	2·23	3·21	3·88	4·32	4·48
2·4	2·32	3·36	4·09	4·56	4·76

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A comparison with the table of pressures on p. 99 will show the effect of the buttressing.

Earth Backing.—The typical arrangement of earth and

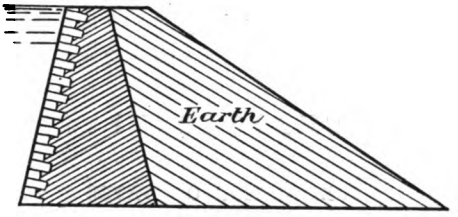

FIG. 39.

masonry adopted for some dams is shown in the accompanying cut. The earth backing tends to prevent expansion and contraction of the masonry.

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CHAPTER VI

PURIFICATION OF WATER

WHEREVER the drainage area of a river is thickly populated the water derived therefrom must be looked on with suspicion, and may be, and almost certainly is, unfit for domestic use without preliminary purification.

The principal methods adopted on a large scale are: (1) Simple deposition in settling tanks, either without the addition of anything to the water, or with the addition of lime, as in Clark's process already mentioned; (2) slow natural filtration through sand; (3) filtration through 'mechanical' filters, with the addition of reagents.

Settling Tanks.—Simple deposition, if allowed to continue long enough, would probably remove nearly all suspended impurities (including bacteria) from dirty water, but the time required for this would be very great. While settlement is going on in any tank, the water must be allowed to remain nearly still, and hence simple sedimentation involves the construction in duplicate of settling tanks capable of holding many days' supply.

About 50 per cent. of the bacteria present can be removed by twelve days' storage, the water not being kept entirely still.

Such tanks are therefore used as very helpful auxiliaries to some system of filtration rather than as a means of complete purification.

Where there is an impounding reservoir this will generally form a most efficient settling tank in itself. There are many cases where streams enter such reservoirs—either natural or artificial—in an extremely muddy state, while the run-off at the lower end is quite clear.

Where, however, the minimum flow is sufficient for the supply—so that an impounding reservoir is unnecessary—if the water be at all muddy, the construction of settling tanks to remove the grosser impurities greatly reduces the cost of subsequent filtration.

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Construction of Settling Tanks.—The capacity of these tanks is generally such as to contain two or three days' supply, and they are best built in duplicate, so that the water in one tank can be drawn off while that in another is settling.

The gross capacity of the tanks should be not less than twelve days' supply, if the first flush of storm water is to be rejected.

They should, where practicable, be provided with a wash-out pipe

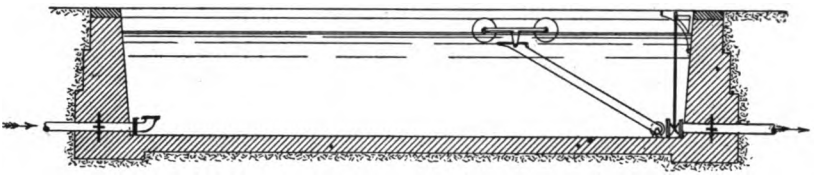

FIG. 39*a*.

at the bottom level, for cleaning, and with means of draw-off at different levels (fig. 39*a*), either by separate valves, or by means of a floating arm, of the construction shown in clearer detail in fig. 39*b*.

The exact shape and dimensions given to these tanks will depend chiefly upon the nature of the ground in which they are to be built, the object being generally to arrive at the most economical design under the given conditions.

Building them entirely above ground involves stronger walls, but

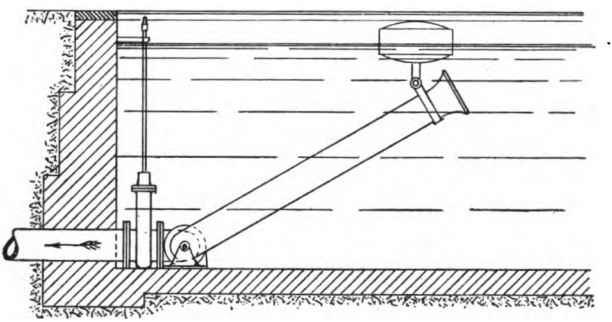

FIG. 39*b*.

saves excavation, gives a greater head, and may simplify the wash-out. A circular shape gives a greater capacity for a given volume of walling than a rectangular one, but will be rather more expensive to build. The construction of the bottom will depend on the foundations. It is probably best, as in other works, to prepare alternative designs for these tanks, and select that which appears to give the best result.

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The banks of large tanks or reservoirs are generally made of earth, battered externally and internally. If built of masonry and vertically, the deposition of mud which takes place on the sides of the sloping banks is avoided.

The walls of small tanks may be built of masonry or of concrete, rendered internally with Portland cement mortar from ½ inch to 1 inch thick, the best proportion being 1½ or 2 of sand to 1 of cement. If used unmixed, the cement expands at so different a rate to the wall, and sets so quickly, that it cracks off. If of concrete, a saving of cost may in many cases be effected by making a lining of glazed bricks, laid alternately, one header course to three stretcher courses, at the back of which the concrete is rammed in. The brick lining should be kept slightly ahead of the concrete, to give the cement mortar in which the bricks are laid time to set, and this lining will then serve as a mould for the concrete, and obviate the necessity of timbering.

Clark's process, where lime-water is added to the water to reduce the hardness, has already been mentioned (p. 11).

An improvement on Clark's process has been introduced by adding also the precipitate from previously softened water, which is in a flaky state, and, in settling slowly down, carries with it the fine particles of lime, thereby much reducing the time required for deposition.

If the hardness be 'permanent,' it may be diminished by adding carbonate of soda.

Sand Filters.—In England, the common method of purification is that of slow filtration through sand.

A 'filter bed' is generally built of masonry or concrete about 8 feet deep, and with its bottom either quite flat or with a very slight regular slope towards the outlet, which may be situated say at A in fig. 40. The outlet is generally made by a pipe passing through the wall at the level of the bottom, and in order to enable the whole of the water to be drawn off, the mouth of this pipe usually enters into a depression in the bottom of the bed; or this depression may take the form of a channel running nearly the whole length of the bed, as shown by the dotted lines. In either case it must be covered with flags laid not quite touching, so as to allow the water to enter, or filled with large stones to prevent it from being choked.

The outlet channels are sometimes made of open jointed stoneware pipes.

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Bottom Layer.—The bottom of the bed is then covered all over in some way with a layer of stones of considerable size, designed to allow the water free passage and prevent choking.

In some beds cross channels or drains have been built from 3 to 6 feet apart, running into the central channel already mentioned, the intermediate space being packed with loose stones; but it has been found that there is a tendency for the water to filter more quickly through the parts of the beds directly over these channels, so doing away with the necessary slow uniform rate of filtration.

What is known as the 'cellular brick floor' was therefore used, the bottom of the bed being covered with bricks (or concrete blocks)

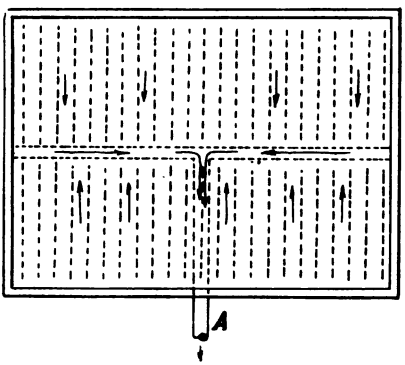

FIG. 40.

laid on edge, in rows 2 or 3 inches apart, and covered over with bricks laid flat and close together, no mortar of any sort being employed.

This gave free passage everywhere for the water, and took up but little height, but was found to fail through the occasional crushing or tumbling down of one of the bricks, which led to a leak in the filter.

The whole bottom in some beds is covered with stoneware drain pipes 3 or 4 inches in diameter laid with open joints, and packed between with coarse gravel.

Gravel.—Whatever be the arrangement of the bottom, it must be covered with at least 6 inches of gravel, carefully screened and arranged in layers of gradually decreasing coarseness, the bottom layer being generally made of stones preferably less than 2 inches and more than 1 inch across, while the top layer should be free from sand but should pass through a $\frac{3}{8}$-inch mesh.

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The total thickness of this gravel bed depends to some extent upon the uniformity of the bottom drainage, but from 6 to 12 inches is generally ample.

Sand.—The gravel is covered with a layer of clean sand. It has been found that, with the usual rates of filtration, all the actual filtering is done in the uppermost few inches of the sand, and hence that, at a minimum, its thickness need not exceed say 10 to 12 inches. But for economic working it is usually made from 2 to 5 feet thick, though probably a thickness of from 20 inches to 3 feet may be taken as giving as good a result as any.

The sand should be clean and sharp, and as nearly as possible pure silica. The grains should be about $\frac{1}{125}$ of an inch in diameter, with but few grains more than ·01 or less than ·005 inch.

Action of Filters.—If water be allowed to filter very slowly through such a bed, the coarser particles of mud, etc., are at once arrested by the upper layers of the sand, and in a little while a thin coating of mud is formed, and a sort of gelatinous vegetable growth sets in at the surface, the whole forming a sort of covering over the sand, which is itself by far the most efficient part of the filter. It is not indeed safe to allow the water to run through at the full rate until this skin is formed. Once formed, however, given a slow rate of flow, it will arrest nearly all the bacteria in the water, and by far the greater proportion of all impurities not actually in solution.

Rate of Filtration.—The rate of filtration allowable necessarily depends to some extent on the previous quality of the water, but in ordinary cases it may be taken that it should not exceed a vertical velocity over all of about 4 inches per hour—equivalent to about 8 cubic feet, or 50 gallons, per superficial foot of filtering area per day. The limiting rates in common use are about 3 to 6 inches vertical velocity per hour; two gallons per square foot per hour is a rate frequently allowed for in England.

Regulation of Rate.—It is evident that as the filtration of water in such a bed proceeds, the pores in the upper layers of the sand will become choked, and the rate at which the water filters through will decrease, unless the head or pressure forcing it through be increased. Hence arrangements must be provided whereby the 'filtering head' (that is, the difference of level between the water inside the bed and at the outlet) may be regulated, so as to produce a constant flow.

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Many methods of doing this have been devised, coming under two heads:—(1) Those in which the level at the outlet is constant, the head being adjusted by altering the level of the water in the bed; (2) those in which the level at the outlet is variable.

Examples of the first sort are shown in fig. 41.

On the left the outlet pipe A is turned upwards inside the service reservoir B, so that the difference of level between its mouth C and the top water level (T.W.L.) of the filter is equal to the maximum filtering head it is intended to allow. The water being allowed to run on to the bed at a given rate, as the pores become choked the level in the bed will rise, so increasing the filtering head and maintaining the rate of flow through the bed. When the top water level is reached, the inlet pipe must be shut down and the filter cleaned, as will be afterwards described. The object of the upturned pipe is,

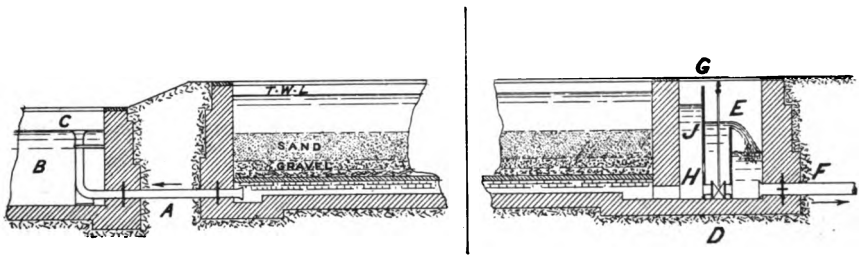

FIG. 41.

of course, to supply a constant back pressure with a variable water level in the reservoir B.

With this arrangement the rate of filtration is fixed entirely by the rate of inlet to the bed. This is regulated by a sluice valve under the control of an intelligent foreman, and the efficiency of the filtration must be tested by examining the filtered water.

On the right the upturned pipe is replaced by a chamber divided by a partition E, over the top of which the water flows to reach the outlet pipe F.

A second partition, G, may be introduced having a submerged orifice H, the size of which can be calculated so as to give the desired rate of flow with any selected small difference of level. The difference of level between the water in the filter bed and that in the compartment J is the real filtering head, while that on opposite sides of the partition G indicates the rate of filtration.

If the water rises too high in the space J, it is running through

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too quickly, and the inlet valve requires to be screwed down, and *vice versa.*

This arrangement is known as a gauging chamber.

It is evident that the filter could not be emptied below the level of the top of the plate E. Hence a valve must be introduced between the two plates at the bottom level for this purpose. The size of the gauging chamber is regulated by the stock sizes of the valves, etc., which have to be introduced in the different compartments.

Variable Outlet Level.—A somewhat similar arrangement has also been used, but without the plate E. The orifice H is nearer the surface, and the plate G containing it is capable of being raised or lowered. The height at which the water stands in the compartment J indicates the rate of flow as before, and if this is too great the plate is raised, thereby diminishing the filtering head, and re-establishing the normal rate of flow.

The catalogues of most manufacturers of hydraulic apparatus show appliances for this purpose.

In general it may be taken that the more complicated the appliances adopted, the more liable they are to get out of order; it is, moreover, doubtful how far an absolutely constant rate of filtration is desirable, as the efficiency with a given rate depends partly upon the state of the filter.

Hence in many waterworks schemes, separate gauging chambers are not provided for each filter, but the rate of supply is measured elsewhere, often at some point between the filters and the source of supply.

It is desirable to provide a separate drawoff from each filter, so that in the event of any breakdown, the effluents can be separately examined to locate the defect.

Cleaning.—When a filter has been at work for some time, the gelatinous coating already referred to (largely composed of fine particles of mud, together with bacteria and animalculæ) becomes so thick that the water is unable to pass through at the normal rate, even with the maximum filtering head.

The water is then shut off, the bed allowed to drain, and the top layer of the sand scraped off. As little is removed as possible—usually from ¼ inch to 1 inch—and it is made into small heaps, which are generally carried in barrows or otherwise to 'sand washers.'

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These are placed in any convenient position near the filters (sometimes in a walled-in chamber in the centre of the bed), and their function is to remove all mud and organic matter by washing, leaving the sand fresh and clean.

Many forms of sand washer are in use, the simplest consisting of a chamber with a sloping bottom, level with the ground at its higher end, and backed by a wall, as shown in plan and section in fig. 42.

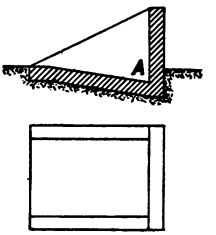

FIG. 42.

The sand is placed at A and stirred by hand while a jet of water is turned on it from a nozzle in front.

Another form consists of a cast iron box with perforated bottom on which the sand is piled, and through which a stream of water rises, carrying with it the lighter particles of dirt and flowing to waste over a weir in the side of the box (see fig. 43), or the arrangement

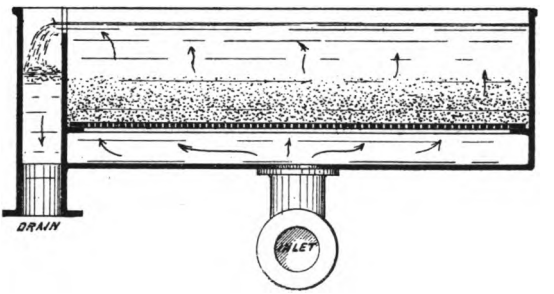

FIG. 43.

illustrated at fig. 44 (Plate I.), which was designed by Mr W. Hunter, M. Inst. C.E., Engineering Director to the Grand Junction Waterworks Co., and consists of a series of small tanks connected by pipes through which the dirty sand is forced from one tank to the next and so on by means of a current of water induced by a smaller stream under a considerably greater pressure. The lighter particles of dirt flow over through suitable orifices to a drain, and the clean sand is finally

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injected into a hopper, whence it is replaced upon the beds by manual labour.

The time elapsing between two successive cleanings varies with the quality of the water, but in England it is generally two or three weeks, although at times it may only be as many days.

Limiting Depth of Sand.—The clean sand is not replaced after each washing. The main reason for this is that the filter acts more efficiently if the sand at the surface is not altogether clean. Hence several scrapings of the surface are made until the original depth of two or three feet of sand is reduced to about ten to twelve inches. Less than this depth is not considered safe, hence when this limit is reached the thickness is made up to the original amount with the washed sand, care being taken to mix the fresh sand well with that already on the bed.

The top layer of the sand remaining after the last scraping, is sometimes removed before putting in the clean sand, and put on top of the latter.

Arrangements are sometimes made for filling the bed backwards from below with filtered water after the clean sand is replaced.

If this be not done, the first water running through the clean filter will be inefficiently purified, and it must be allowed to run to waste until the water is found by examination to be sufficiently pure, or until the time found necessary for this has elapsed.

Maximum Depth of Water.—The maximum depth of the water over the sand is in pretty close agreement, as a rule, with the maximum filtering head. How much this should be, is a matter about which there has been some difference of opinion, but from 2 to 3 feet may be taken as giving the best results. A greater pressure is liable to cause breakages and leaks.

Total Depth of Bed.—Exclusive of the bottom wall, the limits of depth according to the figures given are:—

Gravel, etc., . . .	·75	to	1·5	feet.
Sand, ...	1·50	„	3·0	„
Water, ...	2·00	„	3·0	„
Total, . .	4·25	„	7·5	„

The higher dimensions are nearer the average than the lower, and many filter-beds have had the different layers much in excess of these figures.

Air Pipes.—It is necessary to provide for the passage of air to

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and from the bottom of the bed otherwise than through the sand, in order to avoid the disturbance of the filtering layers. For this purpose vertical air pipes, passing through the sand, may be used, or a ventilating passage left in the wall itself.

Roofing of Filters.—The filters are sometimes roofed over. To ensure efficiency it is desirable that the water should not freeze at all, and the roofing tends to prevent this. It also keeps the water at a more even temperature. Direct sunlight may cause a considerable evolution of gas from the organic layer at the top of the sand, which sometimes leads to the lifting of portions of the gelatinous coating already referred to. These rise to the surface, floated by the evolved gas, and leave what amount practically to temporary leaks.

The collecting or gauging chambers (if any) must, of course, be covered, as they contain filtered water.

Paired Filters.—Filter beds have been arranged in pairs, with one collecting chamber common to both. This system, however, has the serious disadvantage that in the event of any inefficiency, it is difficult to locate the defect, and it may be necessary to close both beds. With a separate drawoff from each filter it is more easy to find out which bed is out of order.

Area required.—It is evident that the cleaning of a bed involves the stoppage of its action for a certain time. Hence it is necessary to provide extra filtering area to allow for this. The ratio of the extra area to that required to be constantly in use, will be the same as that of the time required for cleaning and starting (including a portion of that required for refilling) to the time a bed can work without cleaning.

This ratio is really a variable one, inasmuch as the quality of the water varies with the weather, hence exact calculation is impossible, and the filters must be strained more at some times than at others.

In general, in England, it may be taken that an extra area of one-fourth to one-sixth of the area in work will be sufficient.

Number of Beds.—Thus, for a supply of 500,000 gallons per day, and a rate of 50 gallons per square foot per day, 10,000 square feet would be necessary. One-fourth extra would be 2500, making 12,500 in all. This would then be divided up into beds of a convenient size; the total number may be some multiple of 5, so that one-fifth the total area may be idle while the remaining four-fifths are at work, as allowed for.

In a supply no bigger than this, the number of beds adopted under

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the conditions named would be most likely five, inasmuch as the fewer the beds the less the cost of walling. But for very large supplies ten or more might be used, as there is a limit beyond which increasing the size of each bed does not lead to economy. This limit is about 30,000 square feet.

Arrangement and Dimensions.—The arrangement of the beds will, in England, generally depend on the ground where they are to be made; but where space is plentiful, the arrangement aimed at should be that which will give the greatest economy of walling, etc., together with facilities for easy working.

Thus each bed should be as near as practicable to the sand washers, and for compactness and economy of wall it is generally sound practice to aim at a square shape for each filter and also for the whole set.

This is only practicable for four, nine, or sixteen beds. In other cases the dimensions must be altered, so as to depart as little from the square shapes as possible.

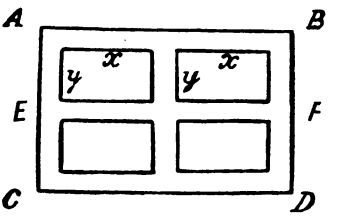

FIG. 45.

Worked Example.—We will work out fully two examples with four and six beds respectively, finding the best dimensions for economy of walling with rectangular shapes.

(1) Suppose four beds are arranged as in fig. 45.

Let x and y be the lengths and breadths of each bed, A the total area, d the thickness of the walls (supposed the same throughout), and k the depth of the bed internally, so that $k+t=$ total depth of vertical walls.

Then volume of walls A B, C D, and E F $= 3.t\ (2x+3t)\times(k+t)$.

" " A C, B D $= 3t\times 2y\times(k+t)$,

and volume of bottom $= 4xy\times t$.

But $4xy = \mathrm{A}$, $\therefore\ y = \dfrac{\mathrm{A}}{4x}$,

and total volume $=$ V (say)

$$= 3t\left(2x+3t+\frac{\mathrm{A}}{2x}\right)\left(k+t\right)+\mathrm{A}t\,.$$

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$$\therefore \frac{dV}{dx} = 3t(k+t)\left(2 - \frac{A}{2x^2}\right)$$

$\therefore$ when V is a minimum,

$$\frac{A}{2x^2} = 2, \text{ or } x = \frac{\sqrt{A}}{2},$$

$$\therefore y = \frac{\sqrt{A}}{2},$$

or the square arrangement is best.

(2) For an arrangement of six beds as shown in fig. 46, using the same letters as before:—

Volume of longitudinal walls $= (k+t) \times 3t(3x+4t)$.

" transverse walls $= 4t(k+t).2y$,

" bottom $= At$,

and $6xy = A \quad \therefore y = \frac{A}{6x}$,

$$\therefore V = t(k+t)(9x+12t+8y) + At$$

$$= t(k+t)\left(9x+12t+\frac{4A}{3x}\right) + At.$$

$$\therefore \frac{dV}{dx} = t(k+t)\left(9 - \frac{4A}{3x^2}\right).$$

But when V is a minimum, $\frac{dV}{dx} = 0$.

$$\therefore 9 - \frac{4A}{3x^2} = 0,$$

or $27x^2 = 4A$,

whence $x = \frac{2\sqrt{A}}{3\sqrt{3}}$,

$$\therefore y = \frac{A}{6x} = A \times \frac{3\sqrt{3}}{12\sqrt{A}}$$

$$= \frac{\sqrt{3A}}{4}.$$

$$\therefore \frac{x}{y} = \frac{2\sqrt{A}}{3\sqrt{3}} \times \frac{4}{\sqrt{3A}}$$

$$= \frac{8}{9}.$$

Hence with this arrangement we get the most economical result when x is eight-ninths of y. A little study will show that the total length A B will be to A C as 4 to 3.

Inlet Appliances.—As the surface of the sand is lowered by

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repeated scrapings, if the inlet be in the first instance arranged at the level of the top of the sand, it will soon be some distance above it, and the water, tumbling down on the sand, will be liable to wash the latter away in places.

To avoid this contingency, an adjustable inlet may be used, consisting of a series of bell-mouths dropped one upon another into the mouth of the inlet-pipe, till the topmost is level with, or very little above, the sand; or preferably, a secondary partition wall may be built across the entire width of the filter, with its upper edge level with the sand at its greatest thickness. The length of the wall is so great in comparison to the depth of water flowing over it when the filter is slowly filled, that the falling water does not disturb the sand to any serious extent. This arrangement is indicated in fig. 47 (Plate II.).

Earth Walls.—Filter beds are often constructed with earth

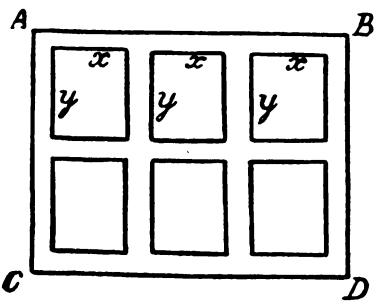

FIG. 46.

walls in somewhat the same fashion as described for impounding reservoirs, though on a much smaller scale.

An example of a set of sand filters on the English or natural filtration system is given in Plate II., fig. 47.

Artificial Filtration.—If a little alum be added to ordinary water, chemical action takes place whereby hydrate of alumina (a gelatinous-looking substance which settles somewhat slowly in water) is formed. This substance, in settling, has been found to carry with it most of the suspended impurities in the water, and also, in some way, to improve the colour.

Moreover, the gelatinous nature of the hydrate of alumina apparently causes it to have the same effect in a filter as the gelatinous covering formed by organic growth in the process of natural filtration already described.

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The difference is that the coating in this case forms at once throughout the whole depth of water, and forms no matter how fast a rate be adopted.

The disadvantage of the method is that sulphuric acid is also liberated in the reaction. In ordinarily hard water this acid combines with the mineral bases in the water, and is thus neutralized. In very soft water it is necessary to add some base for this purpose.

In America 'mechanical' filters based on this principle have been somewhat extensively tried, one form consisting of a closed metal tank into which the water is pumped after addition of the alum, and which contains the sand for supporting the aluminium hydrate.

A rate from 50 to 300 times in excess of that used in natural filtration has been found possible, so far as the removal of mineral impurities in suspension is concerned. It is doubtful if any chemical or biological change is produced.

To clean the filters, water is forced upwards through the sand, requiring about 5 per cent., at each cleaning, of the water filtered between two cleanings.

The depth of sand is about 30 inches.

Full accounts of the tests made with these filters will be found in the treatise on *Waterworks Engineering* by Prof. A. Prescott Fowler.

The quantity of alum required is about 2 grains per gallon.

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CHAPTER VII

SERVICE RESERVOIRS

It has already been stated that the water supplied to a town is not used at a uniform rate, either daily or yearly.

The maximum rate of demand takes place when there is a large fire to be extinguished. What the actual rate of use is under these conditions then, will depend upon the extent of the fire and the facilities for obtaining and using water. It must be calculated from the size of the fire-hose and the number likely to be in operation at once.

To this must be added the greatest rate of supply for ordinary purposes, and the total will be the maximum rate.

This will last only until the fire is extinguished. The greater part of the daily supply is, in any case, used during the earlier hours of daylight.

Filter beds are designed to work at a uniform rate, whence two courses are open:—

(1) To so design the filters that they can pass the water at the maximum rate required, some of them being idle at other times.

(2) To provide clear water reservoirs to store that which is filtered during the hours of small demand, for use when the requirements become greater.

It is very seldom that a water-supply scheme is designed as in (1), without clear water reservoirs. In some cases where the water is fairly pure before filtration, a 'bye-pass' or connecting pipe has been introduced between the supply main leading to the town and the pipe from the source to the filters, thus enabling the unfiltered water to be passed on to the town in cases of sudden demand, as for fires, etc.

The system, however, has the disadvantage that every time the

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connection is made, the town pipes become filled with dirty water, and it is seldom used in England, and not commonly anywhere.

Clear Water Reservoirs.—In nearly all water supply schemes clear water reservoirs are provided.

In addition to diminishing the necessary filtering area, and holding a reserve for fire extinction, they serve another important purpose, especially in large towns.

If the whole of the water delivered to such a town be carried to the consumers through one or more main pipes emanating from a single point, and subsequently branching off in other directions, there will be a great strain put on these mains during the hours of greatest consumption, and they will have to be of relatively large diameter to deliver the water to the more remote parts of the town under sufficient pressure when water is also being drawn from them all along the route.

But by erecting several clear water reservoirs at different parts of the town, these can be slowly filled through relatively small pipes during the hours of slow consumption, and serve as 'centres of distribution' from which the water can be supplied to the consumers through smaller mains and at a much more uniform pressure, always provided that suitable positions for the institution of such reservoirs exist.

Where there are several such reservoirs, they should be connected with one another, so that one can be filled from the others if necessary, and a 'bye-pass' should connect the main from each reservoir with the pipe joining it to the next, so that if one has to be emptied for cleaning or other purposes, the supply to the district it serves may be carried through this bye-pass.

Where there is only one clear water or 'service' reservoir, it should be built in two compartments for the same reason, and should be in as central a position as practicable.

It is frequently built near the filter beds.

Capacity of Service Reservoirs.—Service reservoirs have been made to contain two or three whole days' supply of water. It is now considered, however, that the capacity should be no greater than necessary, in order to diminish the chances of stagnation or development of bacterial life in the water after filtration.

The absolutely necessary volume amounts to seven or eight hours' supply, together with the quantity allowed for fire extinction.

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The latter, as has been said, is variable. It depends upon the construction of the houses, the size of the town, etc.

Moreover, in some countries (as in America) more water is used for fires than in England.

In general, however, it may be taken that about one day's supply will be necessary for towns up to about 10,000 inhabitants, and that above that the capacity may be less, but should never fall below half a day's supply.

Construction.—Service reservoirs must of course be covered in all cases where they are near large towns, and they should be covered everywhere.

In England they are most commonly built with a considerable portion of their depth below ground, the excavated earth being banked round the upper parts of the walls. The roof is built of concrete or masonry, so that a covering of earth may be put on it

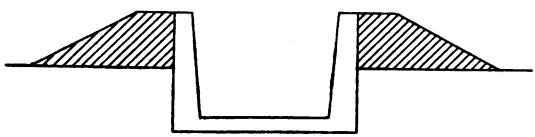

FIG. 48.

also, the object being to keep the water at an even temperature, as well as pure.

The selection of suitable dimensions is governed by the same considerations as laid down for filter beds.

The depth is generally from about 9 to 15 feet, increasing with the capacity.

For mere economy of walling, a greater depth would give a better result in most cases, but the cost of the excavation and embankment (supposing the reservoir made as in fig. 48) increases with the depth.

Neglecting this, suppose the reservoir square in plan and of capacity v.

Let h = depth in feet, and suppose the bottom to be 2 feet thick and the vertical walls 18 inches at the top with a batter of 2 in 12.

Let x = side of base in feet.

Then

$$x^2 \times h = v, \quad \therefore x = \sqrt{\frac{v}{h}}$$

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And volume of masonry in walls

$$=x^2\times 2+4(h+2)\left(\sqrt{\frac{v}{h}}+1\right)\left(1{\cdot}5+\frac{h}{12}\right) \text{ cubic feet.}$$

$$=\frac{v}{h}\times 2+\frac{h+2}{3\sqrt{h}}(\sqrt{v}+\sqrt{h})(18+h).$$

If the roof be taken as costing twice as much as the bottom, this becomes altogether

$$\frac{6v}{h}+\frac{1}{3}\left(h^2+\sqrt{v}h^{\frac{3}{2}}+20h+20\sqrt{v}h^{\frac{1}{2}}+36+36\sqrt{v}h^{-\frac{1}{2}}\right).$$

Differentiating and equating to zero,

$$36v=2h^3+3\sqrt{v}.h^{\frac{5}{2}}+20h^2+10\sqrt{v}.h^{\frac{3}{2}}-18\sqrt{v}.h^{\frac{1}{2}}.$$

If any value be given to v, h may be found by trial from this equation, and gives the height which would give the minimum quantity of masonry under the conditions stated.

Thus if $v=24{,}000$ cubic feet (150,000 gallons) it will be found that $h=20$ feet nearly, giving for the base a square of about 35 feet wide.

If we neglect the first and third terms on the right of the equation (which are relatively unimportant), it becomes

$$36\sqrt{v}=h^{\frac{1}{2}}(3h^2+10h-18).$$

The reservoir would in practice be made rather more shallow in consequence of the extra cost of construction.

The cost of the roofing, per square foot, also increases somewhat with the depth, in consequence of the greater size of the pillars required to support it.

The best plan is, probably, to make two or more rough plans in the first instance, giving alternative designs, and select that which gives the best result.

Roof.—The roof is supported by the side walls, and generally by pillars resting upon stone slabs let into the bottom, the latter being thickened below each pillar so as to provide a sufficient foundation.

These pillars may be of masonry, or cast iron columns. If of masonry they must be rendered with cement up to at least 6 inches above top water level, so as to prevent the possibility of water getting inside the rendering when the reservoir is full, and causing an outward pressure when the level falls.

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The roof may be finished in various ways. A simple construction requiring little height is to run girders across in one direction from pillar to pillar, bolted to the tops of the pillars, and cross girders in the transverse direction, not more than about 3 feet 6 inches apart, and supported by the main girders (fig. 49).

Boarding is erected underneath these, and the space between them is carefully filled in with concrete, rendered over with bitumen to make it watertight, and having, if desired, a slight slope towards one side for draining off the surface water.

When the concrete is thoroughly set, the boarding is removed, and a covering of earth is put over the concrete, usually 6 to 12 inches deep.

The distance apart of the pillars is generally about 10 to 15 feet, and the cross girders are from 3 to 5 feet apart.

The thickness of the concrete should be such that if a beam 1 foot wide, such as A B, be supposed separated from the rest, it will be strong enough to bear its own weight—together with that of the earth covering and a load of about 120 lbs. per square foot to allow for moving load,—without the maximum tensile stress exceeding 2 tons per square foot.

Example.—Thus suppose the cross girders are 4 feet apart. Try 10 inches thickness.

$$\text{Volume of beam} = 4 \times \frac{10}{12} = 3\tfrac{1}{3} \text{ cubic feet.}$$

$$\begin{aligned} \text{Weight of concrete} = 3\tfrac{1}{3} \times 130 &= 440 \text{ lbs.} \\ \text{Weight of gravel 9 inches thick} = 4 \times \tfrac{3}{4} \times 110 &= 330 \text{ ,,} \\ \text{External load} = 4 \times 120 &= 480 \text{ ,,} \\ \text{Total} &= 1250 \text{ lbs.} \end{aligned}$$

$$\text{Bending moment at centre} = \frac{wl}{8} = \frac{1250}{2} = 625 \text{ ft. lbs. units.}$$

$$\text{Moment of resistance} = \frac{\text{breadth} \times \text{depth}^2 \times \text{maximum stress}}{6}$$

$$= \frac{s}{6} \times \left(\frac{10}{12}\right)^2.$$

$$\text{Equating, } s = \frac{625 \times 6 \times 12^2}{100} = 5400 \text{ lbs. per square foot.}$$

This is excessive.

Working in the same way, but with a 12-inch thickness, we obtain $s = 3990$ lbs. per square foot, which is safe.

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Other forms of Roof.—Instead of the cross girders, concrete or brick arches frequently form the roof, supported by the main girders, which are held in place by tie-rods, as shown in Plate III.

Or in places where iron girders are expensive, narrow masonry arches may be built at a lower level from pillar to pillar in one direction. Vertical walls built on these arches form the abutments for the continuous cross arches which form the roof.

Ventilation and Manholes.—To equalize the air pressure and allow for ventilation, ventilators must be placed in the roof. Stock forms will be found in the catalogues of all makers of waterworks appliances.

A manhole must also be constructed to allow access to the inside, for cleaning out, attending to leaks, etc.

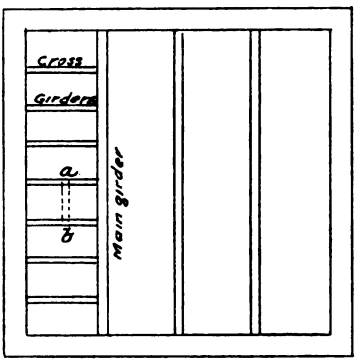

FIG. 49.

An example of a small covered service reservoir is given in the accompanying Plan, Plate III.

The walls are of 4 to 1 concrete, lined with blue brick, chiefly in stretcher courses with occasional headers.

The bottom is of concrete thickened under the roof columns and rendered all over with Portland cement mortar at 1 to 1, and with a slight fall towards the wash out valve. The concrete roof is carried by steel joists supported on cast iron columns and stayed with wrought iron tie bars. Material excavated from the lower portion is used to form a covering over the whole.

Entrance is effected by a manhole at the head of a flight of steps up the bank, and ventilators are fixed at intervals in the roof. The wall at one end is made of a greater thickness than the rest to allow of it becoming a partition wall in case of future extension of

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the reservoir. The inlet, outlet, and overflow and wash out pipes are clearly shown. A bye-pass is provided between the inlet and outlet mains to enable the supply to be kept up in case the reservoir requires to be laid off for cleaning, etc. The overflow and wash out pipe is carried in stoneware for a considerable distance until the levels permit of a free outfall being obtained into a stream. The ground in which the works are constructed is a stiff clay of such quality that no puddle is required behind the walls and the whole is perfectly watertight.

Water Towers.—It not infrequently happens that there are certain portions of a town which are situated at too high a level to be supplied from the service reservoir, such portions being usually in the immediate neighbourhood of the reservoir itself. It is customary in this case to provide what is known as a water tower for the high service, consisting as a rule of a brickwork or masonry tower, carrying a tank of cast or wrought iron at an elevation sufficient to supply the highest house in the vicinity. The tank on the tower is filled at intervals from the storage reservoir or by pumping, as the case may be, and is in effect a service reservoir on a small scale. It is sometimes found with a gravitation supply that the pressure in the mains is only sufficient to fill the high tank at night, when the draft on the mains is less than in the daytime. Should this be so, it is convenient to provide a self-closing float valve to close when the tank is full, and a reflux valve on the service main opening outwards, which will be closed while the pressure is higher than the top water level in the tank and permit the latter to fill, but which will open and allow the accumulated water to pass to supply when the pressure falls. Another reflux valve must be inserted, closing against the lower part of the district, just above the point to which a supply can at all times be given, so as to confine the water from the high tank to that part of the district for which it is required.

The tank is sometimes entirely enclosed within the tower, a method which is preferable from the æsthetic point of view, and as a protection from frost, but is usually, for the sake of cheapness, merely set upon the top of the tower, and covered with a light roof. It may be square or round, the former shape having the disadvantage that stays are necessary to support the flat surfaces, while the round tank, although occupying practically the same ground space, must be deeper if it is to hold the same quantity of water. The round

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tank has the advantage of a better distribution of weight upon the foundations than can be obtained with the rectangular shape. A form which requires no staying or supporting girders is a cylinder of wrought iron or steel with a hemispherical bottom, supported on a circular tower by brackets at suitable intervals. If the diameter be considerable, the bottom may be in the form of a shallow segment of a sphere, provided with a ring at its junction with the cylindrical portion, suitably proportioned to take the compression caused by the bottom in its effort to assume the form of a perfect hemisphere.

In many instances high service tanks are supported on a series of cast iron columns braced together in a suitable manner, and furnished with suitable ladders or staircases for convenient access to the tank.

It is advisable where the tank can only be filled at considerable intervals and is liable to be at times almost empty, to furnish a second tank which may be always kept full, in order that in case of fire there may be a sufficient quantity of water for immediate use, and until a further supply can be obtained from the pumping station or other source.

Standpipes.—Another device for supplying a high district or keeping a constant head on the pumping engine, is to pump the water through a pipe rising to the necessary height in the form of an inverted U. In this case the only storage which is available between the periods of pumping is that provided by cisterns in the houses supplied, and the method is not to be recommended unless the cost of an elevated tank should be prohibitive. A substitute for the standpipe system, but one which is open to the same objection, is to pump direct into the high level district and allow the water not there required to escape to the low level only through a valve loaded with such a weight that before it will lift, the highest house cisterns shall have been filled.

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CHAPTER VIII

THE FLOW OF WATER THROUGH PIPES

General Remarks.—It is an unfortunate fact that the formulæ relating to the flow of water appear, in the minds of many who have little time to devote to the study of mathematics, to have become enveloped in a sort of mystery, greater than that which surrounds most of the formulæ of practical utility.

The fundamental theoretical formulæ, however, present very little real difficulty, and the writer is of opinion that the confusion arises mainly from two sources.

In the first place, in this, as in all other cases of motion, frictional and other resistances exist, and the effect of these can only be determined by experiment. From many causes the necessary experiments are difficult and costly, and different observers, in their efforts to find expressions for these resistances, have arrived at somewhat different results. Some books give the results of one observer, and others give those of another, while some again give all the results, and leave the engineer to choose for himself, a matter of obvious difficulty to the practical man.

In the second place, different writers seem, in many cases, to give the formulæ in the units which they consider most likely to be useful. However commendable this may be, it seems to the writer to lead to further confusion, inasmuch as all do not consider the same units the most suitable. Thus one learns a formula giving the flow of a stream in miles per hour, the gradient being in feet per mile. On opening another book, a formula is met with which appears totally different, but which really may be exactly the same formula in different units. It is considered best, therefore, to

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commence the subject at the beginning, and to briefly indicate the theoretical considerations from which the formulæ are deduced, and to put down some of the experimental results for the coefficients of resistance.

Horizontal Pipe issuing from Reservoir.—Suppose a horizontal pipe, C D (fig. 50) to lead from a reservoir in which the water level is maintained at A B. By a well-known theorem, *if there were no loss due to friction or other causes*, the water would flow out through the full area of the pipe with a velocity equal to that it would acquire in falling through the height B C under the action of gravity, or $\sqrt{2g \times BC}$. No matter how long the pipe, the water would issue from the farther end of it with this velocity.

There would be no acceleration, as the whole pressure would be taken up in imparting the initial velocity, and the water behind be moving at the same rate.

As a matter of fact, however, there are frictional resistances which

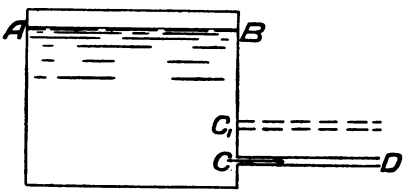

FIG. 50.

cause a gradual retardation of the velocity. This retardation is transmitted to the liquid behind, and the whole being regarded as practically incompressible, will move at the same rate, but slower than with a frictionless pipe.

The result will thus be equivalent to a frictionless pipe starting from some higher level, C_1, and $C C_1$ is the *loss of head* due to friction.

We proceed to inquire into the amount of this.

Laws of Fluid Friction.—The laws which govern the friction of fluids are not the same as for the relative motion of solid bodies.

In the case considered, it is evident that, with the same velocity, the loss of head would be proportional to the length, as the friction in each foot of the pipe is the same.

Variation with Velocity.—Hence it is clear, in the first place, that the friction must depend considerably on the velocity.

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For suppose, with 100 feet of pipe, the loss of head C C_1 were one-tenth of B C, then, if the friction were independent of velocity and the pipe were made ten times as long, the friction, and therefore the loss of head, would be ten times as great, or equal to the whole head B C.

The flow would therefore cease, whereas experience shows that practically it would not do so.

Experiment shows that, for very small velocities, the friction is nearly proportional to the velocity, other things being equal, while for the velocities generally obtaining in practice, it is nearly proportional to the *square*. The proportion does not appear, however, to be exact in either case.

Other Factors.—Again, friction between solid bodies is proportional to the pressure. For liquids this has been shown not to be true, and fluid friction is independent of the pressure.

It is also naturally proportional to the area in contact, and it depends upon the nature of the solid surface.

Now let d = diameter of pipe,

l = length of pipe,

v = velocity of flow,

f = a coefficient depending on the nature of the surface, being the amount of friction on unit area of surface with unit velocity.

F = frictional resistance.

Then the area in contact is πdl, and we may write approximately,

$$F = f.\pi dl.v^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Reduction to Loss of Head.—This expresses the frictional resistance as a force.

To reduce it to a head C C_1, we must use the principle of work.

Suppose all units to be in feet, lbs., and seconds, and let w = weight of unit volume of water, H = head B C. In one second the quantity of water flowing through is $\frac{\pi d^2}{4}.v.w.$ lbs.

This has, in effect, fallen through the head H, or B C.

$\therefore$ loss of potential energy $= \frac{\pi d^2}{4}.v.w.H.$

The kinetic energy acquired by the water is

$$\frac{\frac{\pi d^2}{4}.v.w.v^2}{2g} = \frac{\pi d^2 w v^3}{8g}.$$

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The work done against friction is Fv. Hence equating loss of energy to work done,

$$\frac{\pi d^2}{4}.vw\mathrm{H} = \frac{\pi d^2.w.v^3}{8g} + \mathrm{F}v,$$

$$= \frac{\pi d^2.wv^3}{8g} + f\pi dlv^3 \text{ by (1).}$$

$$\frac{wd\mathrm{H}}{4} = \frac{dv^2w}{8g} + flv^2,$$

$$\therefore\ flv^2 = \frac{wd}{4}\left(\mathrm{H} - \frac{v^2}{2g}\right);$$

but $\frac{v^2}{2g}$ is the head B C_1, which would have given the velocity v in a frictionless pipe.

$$\therefore\ flv^2 = \frac{wd}{4}.(\mathrm{H} - \mathrm{BC}_1),$$

$$= \frac{wd}{4} \times \mathrm{CC}_1$$

$$\therefore\ \mathrm{CC}_1 = \frac{4flv^2}{wd} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

This gives the head, in feet, lost by friction, in terms of the actual velocity of flow, the length and diameter of the pipe, and the co-efficient f, which is only determinable by experiment (*vide* p. 138).

Total Head for Given Velocity.—The total head B C is found by adding B C_1 to the above, the latter being the head which would have given the same velocity in a frictionless pipe.

$$\text{Hence } \mathrm{BC}_1 = \frac{v^2}{2g},$$

$$\therefore\ \mathrm{BC} = v^2\left(\frac{1}{2g} + \frac{4fl}{wd}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

This gives the total head in feet where

v = velocity of flow in feet per second.

g = acceleration due to gravity in foot second units = 32·2 nearly.

f = friction in lbs. on 1 square foot of the pipe surface, with a velocity of 1 foot per second (*vide* p. 130).

l = length of pipe in feet.

d = diameter of pipe in feet.

w = weight in lbs. of 1 cubic foot of water.

It is to be understood that this formula gives the head requisite

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merely to cause the water to flow through the given pipe, and out at the end, with the velocity v.

No loss but that due to friction in the pipe is here taken into account, and the pipe is still supposed to be horizontal.

Other Losses.—The only other loss, in the case considered, would be due to the sudden entrance of the water from all sides into the mouth of the pipe at C (fig. 51). This tends to produce a contraction of the issuing stream, practically equivalent to a further reduction of the velocity all along the pipe, and hence a further loss of head.

The amount of this loss depends on the shape of the junction, and will be reverted to on p. 151.

Velocity due to given Head.—In the equation last given, put the total head BC = H, and we have

$$H = v^2\left(\frac{1}{2g} + \frac{4fl}{wd}\right).$$

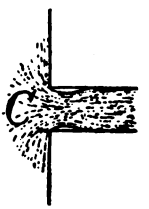

FIG. 51.

Now put $g = 32{\cdot}2$, $W = 62{\cdot}5$, and we obtain

$$H = v^2\left({\cdot}0311 + {\cdot}064\frac{fl}{d}\right) \quad . \quad . \quad . \quad (4)$$

This enables us also to find the velocity due to a given head H.

$$\text{For } v^2 = H \div \left(\frac{1}{2g} + \frac{4fl}{wd}\right),$$

$$= \frac{2gH.wd}{wd + 8fgl} \quad . \quad . \quad . \quad . \quad (5)$$

Substituting again for g and w, this becomes

$$v^2 = \frac{4025.dH}{62{\cdot}5d + 257{\cdot}6fl} \quad . \quad . \quad . \quad (6)$$

Loss in Velocity.—It is to be noticed that with no frictional loss the square of the velocity would have been $2gH$.

Hence the multiplier $\frac{wd}{wd + 8fgl}$ in equation (5) above, gives the ratio which the *square* of the actual velocity bears to that of the velocity

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which would have been obtained in an equal but frictionless pipe under the same head.

In fact we have,

Velocity with frictionless pipe $= \sqrt{2gH}$,

$$\text{actual velocity} = \sqrt{2gH}\sqrt{\frac{wd}{wd+8fgl}},$$

$$\therefore \text{loss in velocity} = \sqrt{2gH}\left(1-\sqrt{\frac{wd}{wd+8fgl}}\right),$$

$$= \sqrt{2gH}.\frac{\sqrt{wd+8fgl}-\sqrt{wd}}{\sqrt{wd+8fgl}} \quad . \quad (7)$$

Variation of Pressure in Pipe.—We have seen, then, that if C K (fig. 52) represent the whole length of the pipe, the water will issue at K with a velocity given by equation (5) and equivalent to that in a frictionless pipe under the head B C_1, where C C_1 is the head lost by friction, given by equation (2), p. 131.

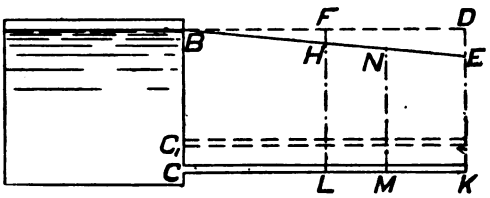

FIG. 52.

In fact, if we make DE = CC_1, the water issues at K with the same velocity as it would under a head E K without friction.

D E represents the loss of head, and K E represents the remaining head or pressure at K, so long as the conditions are unchanged and the water is flowing at the given velocity.

If we consider a point L midway between C and K, half the head D E is lost in the length C L, and the remaining half in the length L K. Hence if FH = ½DE, L H will represent the head or pressure at L. And, in fact, if we join B E, the ordinate M N to this line at any point M will represent the pressure in feet of water at that point.

Hydraulic Gradient.—The slope of the line B E (that is, the value of $\frac{DE}{BD}$) is called the 'virtual slope' or 'relative fall,' and the line B E is called the 'hydraulic gradient.' Its slope evidently represents the loss of head by friction per foot length of pipe. We can

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therefore find an expression for it by dividing the value of D E or C C_1 (equation (2), p. 131) by the length l. That is,

$$\text{Slope of hydraulic gradient} = \frac{4fv^2}{wd} \quad . \quad . \quad . \quad . \quad . \quad (8)$$

It is to be distinctly understood that the slope is unaltered by continuing the length of the pipe, if all other factors remain constant; but if anything else be altered, the hydraulic gradient will also change.

If, for instance, the velocity be checked by a resistance such as a valve at K, the slope will become less. If a pipe of larger diameter be inserted, the loss of head will also be less, and therefore the slope less.*

Conversely, the removal of a resistance at K so as to increase the velocity, or the substitution of a smaller pipe, would increase the slope of the line B E, and diminish the pressure at K.

Effect of other Losses.—Consider now the effect of other

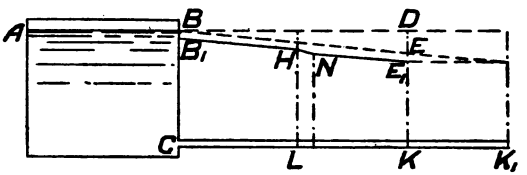

FIG. 53.

sources of loss. The first will be at the point C, fig. 53, where the pipe leaves the reservoir. The contraction of the 'stream lines' there will result in a smaller velocity of exit at K than would otherwise have been obtained. This is equivalent to a sudden fall in the line B E, which marks the hydraulic gradient, shown at B B_1. Similarly if there be a bend at L, the pipe taking there some other direction, the frictional resistance round the bend will be greater than in the straight pipe, and the gradient H N will be steeper at that point.

We thus get B_1 E_1 as the resultant gradient, and E E_1 is the loss of head or pressure due to causes other than friction in the

* This is not obvious at first sight, because the velocity increases with a greater diameter, and hence the loss of head might he supposed to increase also.

But from equation (5), p. 132, $\frac{v^2}{d} = \frac{2ghw}{wd + 8fgl}$.

Hence $\frac{v^2}{d}$ *decreases as* d *increases*, and therefore the hydraulic gradient decreases also, from equation (8).

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straight pipe. We will give formulæ later from which E E_1 could be computed.

It is to be noticed that the final loss of head D E_1 is the same as would have resulted from a longer straight pipe without other losses. K K_1 in the figure shows the extra length equivalent to the other losses considered.

It is a common and convenient practice to reduce other sources of loss to equivalent lengths of straight pipe, and we will revert to this later (p. 153).

It must be borne in mind that the amounts of such losses depend upon the total head B C, and the dimensions of the pipe, as these will affect the velocity of the flow.

Case of a Sloping Pipe.—If the pipe be not horizontal, this

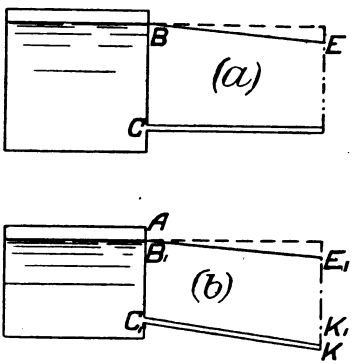

FIG. 54.

will make no difference whatever in the hydraulic gradient, except in so far as the slope of the pipe affects the velocity of flow.

Thus a pipe laid as in fig. 54 (*b*), would give a greater velocity of flow, and hence a steeper hydraulic gradient than one laid as in (*a*), other things being equal.

But if the bead B_1 C_1 be less than B C, so as to give the same velocity with the same dimensions of pipe, then the gradient B_1 E_1 will be exactly the same as B E, and the two lines will be parallel, if the slope of the pipe is not so great as to give an appreciable difference between the length of C_1 K and the horizontal distance C_1 K_1.

Effect on Pressure.—With a pipe laid in as in fig. 54 (*b*), then, the ordinate K E_1 represents the pressure or head at K with the given velocity. It is evident that this pressure will be greater

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than $B_1 C_1$ if the slope of the pipe be downwards, and steeper than $B_1 E_1$.

If the pipe be parallel to $B_1 E_1$, the pressure will be everywhere the same as $B_1 C_1$, and if its downward gradient be less than $B_1 E_1$ (or if it be sloping upwards), the pressure K will be less than $B_1 C_1$.

Modification of Formulæ Required.—By an investigation similar to that already given, it is easy to show that the formulæ already given hold good for this case, except that the head H in the formula must be understood to mean the difference in level between the end K of the pipe and the water surface A, after deducting the loss of head due to other causes than friction in the straight pipe. Or, instead of this deduction, we may express these other losses as equivalent lengths of straight pipe (p. 135). Add the result to the actual length, and take H as the whole difference of level from A to K.

FIG. 55.

The *true* length of the pipe should be taken for l, but this seldom differs appreciably from the horizontal distance.

Case of no Initial Pressure.—Suppose, now, that the water in the reservoir is maintained just above the mouth of the pipe. Then the water will issue at K_1 (fig. 55) with a velocity which can be found by substituting the drop $D K_1$ for H in equation (5), p. 132, or in equation (6).

That is, using the same letters as before (p. 131),

$$v^2 = \frac{2gwd\mathrm{DK}_1}{wd + 8fgl}.$$

From this we can find the head D K required to give any velocity v. For

$$\mathrm{DK}_1 = \frac{v^2(wd \times 8fgl)}{2gwd}.$$

$$= \frac{v^2}{2g} + \frac{4v^2fl}{wd}.$$

To find the actual head required, losses due to bends, etc., would have to be added to this.

Hydraulic Gradient.—Since the water simply issues at K with the given velocity under atmospheric pressure, the hydraulic gradient

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line is clearly parallel to the pipe, and, in fact, runs along its upper surface.

Hence the slope of the line, or the rate of loss of head, is found by dividing the value of D K_1 by the length.

That is, rate of loss of head

$$= \frac{v^2}{2gl} + \frac{4v^2f}{wd}, \text{ dividing the value of } DK_1 \text{ by } l.$$

This is greater than the gradient with a horizontal pipe and initial head (*vide* p. 134) by $\frac{v^2}{2gl}$. If the length be great, this is small by comparison with the second term.

If there is to be a pressure greater than the atmospheric at K—

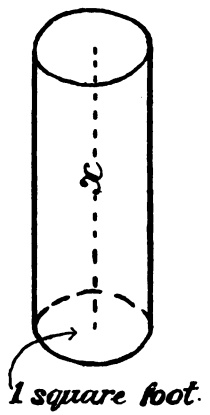

FIG. 56.

as, for instance, if the water has to be raised there to a level K, representing, say, a house cistern—then if other things remain the same, the velocity will be less. To maintain the same velocity, the hydraulic gradient would have to be parallel to its former direction, or the reservoir level would have to be raised to C_1.

The slope of the line C_1 K represents now the hydraulic gradient; C K_1 represents the pipe.

Summary.—It will thus be seen that to find the head required at C, the source, above the level of K, the point of exit, we must find (1) the head necessary to give whatever pressure may be required at K; (2) the head lost by friction in order to maintain the required velocity in the pipe. The velocity is settled by the quantity to be delivered, a suitable diameter being selected. This head is then

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found by equation (2), p. 131; (3) the head required to *produce* the required velocity, independently of friction. This is simply $\frac{v^2}{2g}$; (4) the extra loss of heads, due to bends, valves, etc., as will be explained later.

The sum total will be the required difference of level.

In general, the last two are very small compared with the first two.

We will now consider the experimental results for the value of f, and reduce the formulæ to numbers.

Coefficient of Friction.—In our formulæ, f represents the frictional resistance in lbs. per square foot of surface with unit velocity.

Now suppose x = head equivalent to this (see fig. 56). Then weight of x cubic feet of water $= f$.

$$\therefore\ x = \frac{f}{w}, \text{ where } w = \text{weight of 1 cubic foot.}$$

Now let y = head required to produce the corresponding velocity of 1 foot per second. Then $y = \frac{1}{2g}$.

$$\therefore\ \frac{\text{head equivalent to friction per square foot at unit velocity}}{\text{head required to produce the unit volocity}}$$

$$= \frac{f}{w} \div \frac{1}{2g} = \frac{2gf}{w}.$$

This ratio is what is known as the coefficient of fluid friction, and is denoted by z.

$$\therefore\ z = \frac{2gf}{w} \qquad . \quad . \quad . \quad . \qquad (9)$$

This being the ratio of two heads, is more convenient in the calculations than the friction in lbs. per square foot, and hence the experimenters aim at giving the values of z instead of the direct value of f.

Summary of Formulæ. — We will now summarize the formulæ, replacing f by z, and introducing some new symbols, afterwards giving the values of z.

Let Q = quantity of water delivered in cubic feet per second.

m = hydraulic mean depth.

= area of section ÷ perimeter of pipe.

$= \frac{\pi d^2}{4} \div \pi d = \frac{d}{4}$ (for a full pipe).

h = loss of head by friction in feet.

i = rate of loss of head per foot length.

$= \frac{h}{l}$

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The other letters are used in the same sense as before, viz.:—

d = diameter of pipe.
l = length of pipe.
v = velocity of flow.
g = acceleration due to gravity.
H = total head required.
w = weight of 1 cubic foot of water.
f = frictional resistance per square foot with unit velocity.
z = coefficient of friction.
$= \frac{2gf}{w}$, whence $f = \frac{wz}{2g}$.

All units are in feet, pounds, and seconds.

Then equation (2), p. 131, becomes

$$h = z \cdot \frac{4l}{d} \cdot \frac{v^2}{2g},$$

$$\text{or} \quad h = z \cdot \frac{l}{m} \cdot \frac{v^2}{2g} \quad . \quad . \quad . \quad . \quad (10)$$

Equation (3), p. 131, gives

$$\mathrm{H} = v^2\left(\frac{1}{2g} + \frac{l}{m} \cdot \frac{z}{2g}\right)$$

$$= \frac{v^2}{2g}\left(1 + \frac{l}{m} \cdot z\right) \quad . \quad . \quad . \quad (11)$$

Losses due to other causes and final head required must be added to produce the total necessary head.

Equation (5), p. 132, gives

$$v^2 = \frac{2g\mathrm{H}}{1 + \frac{l}{m} \cdot z} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Rate of Loss of Head.—Now from equation (10)

$$i = \frac{h}{l} = \frac{v^2}{2g} \cdot \frac{z}{m} \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

This enables us to find the rate of loss of head, and hence the slope at which a pipe of given diameter must be laid to maintain a given velocity without loss of pressure.

Velocity.—From this $v^2 = \frac{2gmi}{z}$

$$\text{or } v = \sqrt{\frac{2g}{z}} \cdot \sqrt{mi} \quad . \quad . \quad . \quad (14)$$

This enables us to find the velocity which will be maintained without loss of pressure in a given pipe at a given slope.

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Discharge and Diameter.—Hence we can readily find the discharge, for

$$Q=\frac{\pi d^2}{4}\cdot v=\frac{\pi d^2}{4}\cdot\sqrt{\frac{2g}{z}}\cdot\sqrt{mi}.$$

Putting $m=\frac{d}{4}$ here, this reduces to

$$Q=\frac{\pi}{8}\cdot\sqrt{\frac{2g}{z}}\cdot d^{\frac{5}{2}}\cdot i^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad (15)$$

$$\text{or } d^{\frac{5}{2}}=\frac{8Q}{\pi}\cdot\sqrt{\frac{z}{2g}}\cdot\frac{1}{\sqrt{i}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (15a)$$

z being known, this enables us to find the diameter necessary to maintain a given discharge with a given slope of pipe.

These last formulæ take into account only the loss of head due to friction.

Now, if the length of the straight pipe be long in comparison with the velocity, initial head, etc., it is clear that the actual velocity of flow will approximate to that which would just be maintained by gravity in the given pipe at the given slope; and the longer the pipe the less will be the relative effect of the initial velocity, and the more nearly will the equations just given be true in practice.

But for an exact result we must start with equation (11), instead of (10), when dealing with questions of total head or true discharge under given conditions.

Other Losses.—It will be seen that losses at bends, entrances to pipes, etc., are proportional to the square of the velocity, and hence to the head required to produce that velocity, or $\frac{v^2}{2g}$.

We can, therefore, for the present, express the sum total of such losses as $R\cdot\frac{v^2}{2g}$, where R is a coefficient depending on the nature of the bends, etc.

Total Loss of Head.—Adding this to the expression for H in equation (11), p. 139, we obtain

$$H=\frac{v^2}{2g}\left(1+R+\frac{l}{m}\cdot z\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

This gives the total *loss* of head, or the difference between the head at the source and the available head remaining at the point of delivery.

Velocity and Discharge.—Hence we have

$$v^2=\frac{2gH}{1+R+\frac{l}{m}\cdot z} \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

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This enables us to find the velocity which would actually be obtained with a given difference of head for a given length and diameter. Hence we can obtain the discharge

$$\text{For } Q = \frac{\pi d^2}{4} \cdot v = \frac{\pi d^2}{4} \cdot \frac{\sqrt{2gH}}{\sqrt{1+R+\frac{l}{m}z}} \quad . \quad (18)$$

Head.—This gives $H = \dfrac{8Q^2(1+R+\frac{l}{m}z)}{\pi^2 \cdot d^4 g}$ . . . (19)

Whence we can find the head necessary to give any required discharge, knowing the length and diameter of the pipe.

Diameter.—If the diameter be required, the solution of the equation is less simple.

Putting $m = \frac{d}{4}$, we get

$$H = \frac{8Q^2(1+R+\frac{4l}{d} \cdot z)}{\pi^2 . d . {}^4 g}$$

$$= \frac{8Q^2\{d(1+R)+4lz\}}{\pi^2 d^5 g}.$$

Hence $d^5.H\pi^2 g - d.8Q^2(1+R) = 32Q^2 lz$.

This equation is not very readily solvable.

We may write it

$$d^5 = d \cdot \frac{8Q^2(1+R)}{H \cdot \pi^2 g} + \frac{32Q^2 lz}{H\pi^2 g} \quad . \quad . \quad . \quad . \quad (20)$$

This can be readily solved graphically as follows:—

Let a curve (fig. 57) be drawn giving values of d^5, with O X, O Y as axes. Now calculate the value of $\frac{32Q^2 lz}{H\pi^2 g}$, and mark this off at A, on O Y.

Next calculate the value of $\frac{8Q^2(1+R)}{H\pi^2 g}$, and add to the previous result.

Set off the sum to scale at B, on the ordinate corresponding to the diameter *one foot.*

Join A B by a straight line produced (if necessary) to meet the curve at C.

The value of the diameter corresponding to the point C can then be read off on the scale of diameters.

The same curve answers for all such equations, and the point C may be found by stretching a fine cord between A and B, without actually drawing the line.

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Thus if $Q = 1{\cdot}75$ cubic feet per second,
$l = 15{,}000$ feet,
$H = 25$ feet,
$R = {\cdot}5$, $z = {\cdot}01$ (see below).

Then $$\frac{32Q^2 lz}{H\pi^2 g} = 1{\cdot}850.$$

This is taken at O A.

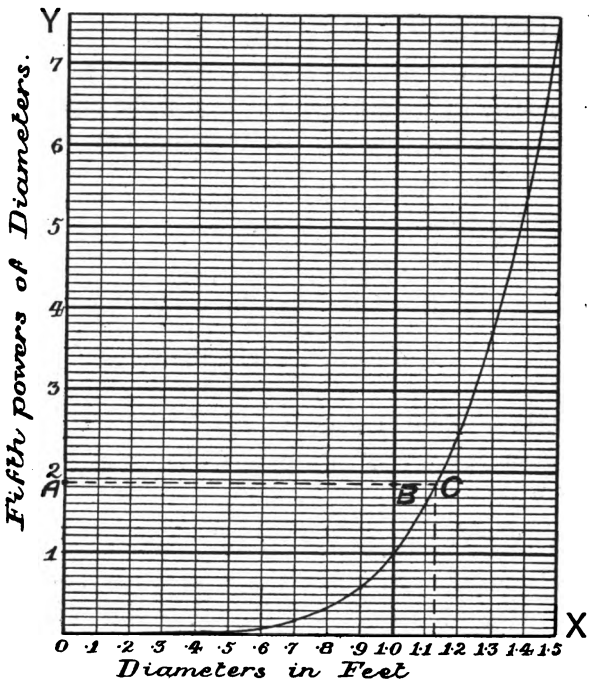

FIG. 57.

Then $$\frac{8Q^2(1+R)}{H\pi^2 g} = {\cdot}0046 = {\cdot}005 \text{ nearly.}$$

$$1{\cdot}850 + {\cdot}005 = 1{\cdot}855.$$

This is taken at 1B.

It is clear that, to the scale of the drawing, it is not sensibly different from O A.

The difference shows the very small effect of the losses other than friction in this case, which is not unlike cases that might occur in practice.

The necessary diameter from the curve is seen to be about 1·13 feet.

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Putting $i = \frac{H}{l}$ in equation (15) (p. 140), and substituting the same values as above, we find $d = 1{\cdot}13$, the same result as before.

As a matter of fact, z is not quite constant for different diameters, and hence, having found d for an approximate value of z as above, we must get the proper value of z from the calculated diameter, and then re-compute the diameter.

Calculations of diameters are seldom required with such accuracy.

Values of Z.—The most reliable experiments for z have been those by M. Darcy. He gives, *with velocities over* 4 *inches per second*, for incrusted pipes, or pipes which have been some time in use—

$$z = {\cdot}00996\left(1 + \frac{{\cdot}084}{d}\right)$$

$$= {\cdot}01\left(1 + \frac{1}{12d}\right) \text{ nearly} \quad . \quad . \quad . \quad . \quad (21)$$

And for new pipes—

$$z = {\cdot}004973\left(1 + \frac{{\cdot}084}{d}\right)$$

$$= {\cdot}005\left(1 + \frac{1}{12d}\right) \text{ nearly} \quad . \quad . \quad . \quad . \quad (22)$$

The equations for z may be written—

${\cdot}005\left(\frac{12d+1}{12d}\right)$, and ${\cdot}01\left(\frac{12d+1}{12d}\right)$ for clean and incrusted pipes respectively.

The diameter is here in feet. Hence $12d$ is the diameter in inches. Therefore, if D be the diameter *in inches*, we have

$z = {\cdot}005 + \frac{{\cdot}005}{D}$ for clean pipes, and $z = {\cdot}01 + \frac{{\cdot}01}{D}$ for incrusted pipes.

The latter is easily calculated with a table of reciprocals. We have merely to take the reciprocal of D, divide by 100, and add ·01.

The coefficient for a clean pipe is half that for an incrusted one.

Unwin's *Hydromechanics* gives a table of values of z.

It is to be remembered that these values are applicable only if the velocity exceeds 4 inches per second.

For a 3 inch pipe, $z = {\cdot}01 + {\cdot}0033 = {\cdot}0133$.
" 6 " " $z = {\cdot}01 + {\cdot}0017 = {\cdot}0117$.
" 9 " " $z = {\cdot}01 + {\cdot}0011 = {\cdot}0111$.
" 12 " " $z = {\cdot}01 + {\cdot}0008 = {\cdot}0108$.

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For clean pipes the values will be half these.

For approximate calculations where the diameter is unknown, z may be taken as ·01. Then calculating the diameter approximately, the correct value of z may be found, and the diameter re-computed more exactly with the new value of z.

But such refinement is seldom necessary in waterworks calculations. The allowance for increase of population is almost always large and uncertain, and the amount of incrustation to which the pipes are liable varies greatly. The margin of error due to these causes is probably greater in most cases than that due to any very small error in the value of z.

Other Values of Z.—Previous experimenters expressed the value of z generally in the form $z=\alpha+\frac{\beta}{v}$, and different observers found somewhat different values for the constants, varying from $\alpha=\cdot0068$ to $\alpha=\cdot0055$, and from $\beta=\cdot0011$ to $\beta=\cdot0014$.

Take $\alpha=\cdot006$, $\beta=\cdot0012$, and substitute in equation (14) (p. 139).

$$\text{Then } v^2=\frac{2gmi}{\cdot006+\frac{\cdot0012}{v}}$$

$$=\frac{2g.m.i.v}{\cdot006v+\cdot0012}.$$

Whence $\cdot006v^2+\cdot0012v=2g.mi.$

Solving this quadratic—

$$v=\frac{\sqrt{(\cdot0012)^2+4\times\cdot0060+2g\,mi}-\cdot0012}{2\times\cdot0060}$$

$$=103\sqrt{mi}-\cdot1, \text{ nearly.}$$

Based on these results, the formula $v=100\sqrt{mi}$ was proposed by Dr Thomas Young, and is still extensively used.

Darcy's Coefficient.—Now take Darcy's coefficients, say for a 6-inch pipe. These are, for a clean pipe, ·0059, for an incrusted one ·0117 (p. 143).

Substitute in equation (14).

For clean pipes,

$$v=\sqrt{\frac{2g}{z}}\cdot\sqrt{mi}$$

$$=\sqrt{\frac{64\cdot4}{\cdot0059}}\cdot\sqrt{mi}$$

$$=104\sqrt{mi}$$

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And for incrusted pipes,

$$v = \sqrt{\frac{64{\cdot}4}{{\cdot}0117}}\ \sqrt{mi}$$
$$= 74{\cdot}3\ \sqrt{mi}$$

It will thus be seen that for an incrusted 6-inch pipe, according to M. Darcy's results, the formula $v = 100\ \sqrt{mi}$ gives a result more than 30 per cent. in excess of the truth.

Quantity and Diameters.—The formulæ for discharge and diameter are given on p. 140.

By substituting for z, when d is known, in equation (15), we can readily find Q.

If Q be known and d required, it is better to take $z = {\cdot}01$, say, in equation (15a).

This gives an approximate value for d. Then from this value calculate the proper value of z (p. 143), and re-compute d if great accuracy is required.

The following table gives Darcy's values for $\sqrt{\frac{2g}{z}}$, and $\frac{\pi}{8}\sqrt{\frac{2g}{z}}$, g being taken as 32·2.

Diameter (inches)	$\sqrt{\frac{2g}{z}}$		$\frac{\pi}{8}\cdot\sqrt{\frac{2g}{z}}$	
	Clean.	Incrusted.	Clean.	Incrusted.
3	98·3	69·5	38·6	27·3
4	101·5	71·8	39·9	28·2
5	103·6	73·3	40·7	28·8
6	105·1	74·3	41·3	29·2
7	106·2	75·1	41·7	29·5
8	107·0	75·7	42·0	29·7
9	107·6	76·1	42·3	29·9
12	109·0	77·1	42·8	30·3
15	109·8	77·7	43·1	30·5
18	110·4	78·1	43·4	30·7
24	111·2	78·6	43·7	30·9

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It will be seen that for clean pipes the velocity $\left(\text{which} = \sqrt{\frac{2g}{z}}.\sqrt{mi}\right)$ will be more than $100\sqrt{mi}$ (Dr Young's formula), so long as the diameter is over 3 inches.

But for incrusted pipes, $v = 100\sqrt{mi}$ gives invariably too high a result.

Diameter.—If we substitute in equation (15*a*), we get

$$d^{\frac{5}{2}} = \frac{8\sqrt{z}}{\pi\sqrt{2g}} \cdot \frac{Q}{\sqrt{i}}$$

$$= \frac{1}{30} \cdot \frac{Q}{\sqrt{i}}, \text{ taking about an average value for } \frac{8\sqrt{z}}{\pi\sqrt{2g}} \text{ from the}$$

table, for incrusted pipes.

$$\therefore d = \left(\frac{1}{30}\right)^{\frac{2}{5}} \cdot \left(\frac{Q^2}{i}\right)^{\frac{1}{5}}$$

$$= \cdot 2565 \left(\frac{Q^2}{i}\right)^{\frac{1}{5}}, \text{ for incrusted pipes.}$$

Taking 42 as the average for $\frac{\pi}{8}\sqrt{\frac{2g}{z}}$ for clean pipes, we get

$$d = \left(\frac{1}{42}\right)^{\frac{2}{5}} \cdot \left(\frac{Q^2}{i}\right)^{\frac{1}{5}}$$

$$= \cdot 224 \left(\frac{Q^2}{i}\right)^{\frac{1}{5}}, \text{ for clean pipes.}$$

All units are in feet and seconds.

A formula commonly given is

$$d = \cdot 23 \left(\frac{Q^2}{i}\right)^{\frac{1}{5}}, \text{ which is about what would be obtained with}$$

a clean 4-inch pipe from Darcy's co-efficients.

Owing to the practical uncertainty as to the amount and nature of the incrustation, as well as the quantity of water to be provided, however, all such calculations contain the elements of large possible errors, and hence it is in most cases sufficient to use the tables given in any pocket book, no matter from which formula they have been calculated.

Still cases will arise which are not treated in the tables, and it is desirable that the engineer should be able to deal intelligently with them.

Mains and Available Head.—The size of main from a storage

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reservoir A to the filters B must be calculated for the difference in level between low water at A and top water at B (fig. 58).

A B represents the hydraulic gradient. Take the slope of this and get the diameter from the tables, according to the discharge required.

Where the pipe is short let h = difference of level A to B. Calculate an approximate value of the diameter as above, and obtain the velocity of flow.

Then from h deduct the head necessary to produce the velocity $\left(=\sqrt{\frac{v^2}{2g}}\right)$ and deduct also the head lost in entrances to pipes, at valves, etc. (p. 152). The remainder, divided by the length, gives the gradient from which the correct diameter is to be obtained.

Strength of Pipe. Pressure at Different Points.—If the pipe sink to a level C between A and B, C D gives the pressure at C, and the pipe must be strong enough to stand that head (*vide* p. 151).

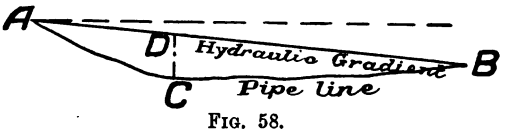

FIG. 58.

If the pipe rise anywhere above the hydraulic gradient, the pressure there will be less than the atmospheric. The pipe acts then as a syphon. Air may be given off at the summit, and interfere with the flow, hence air valves must be placed there, through which such air may be allowed to escape.

Similar remarks apply to the pipe from the filters to the service reservoir.

Distributing Mains.—In calculating the pipes from the service reservoir to the consumers, it must first be borne in mind that the water is not used uniformly.

It is not unusual to work on the basis that the maximum rate of consumption is double the average rate, though in some cases the maximum demand does not exceed the average by more than 50 per cent., due probably to a large and relatively constant trade supply, or to the existence of excessive and constant waste.

Thus, with a town of 7500 inhabitants using 25 gallons (=4 cubic feet) per head per day, the average rate will be

$$4 \times 7500 = 30{,}000 \text{ cubic feet per day,}$$

or ·347 cubic feet per second.

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The maximum rate will then be

$$2 \times \cdot 347, \text{ or say } \cdot 7 \text{ cubic feet per second.}$$

Then suppose A be the position of the reservoir in plan (fig. 59) representing some of the pipe lines, also in plan, and suppose the diameters of the pipes have been provisionally fixed.

Pressure in Mains.—At every point of consumption, there should be a pressure in the main sufficient to take the water to the top of the highest house.

What the pressure should be is a matter on which there is a difference of opinion, but it may be taken that 100 feet of water is a quite sufficient minimum in ordinary cases.

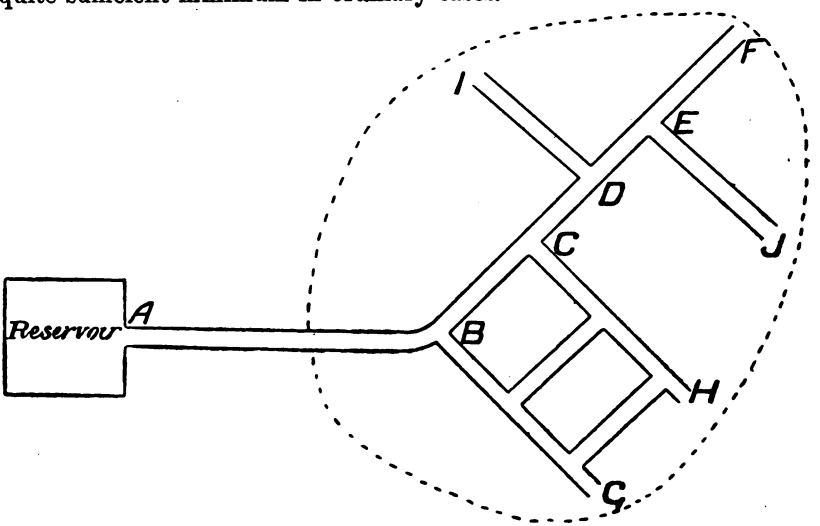

FIG. 59.

Now suppose there is a gradual fall from A to B, and level from B to F, with a fall on the right towards G and J, and a rise on the left towards I.

Then, knowing approximately the number of inhabitants to be supplied through the pipe E F, we can find the maximum rate of flow through there. Then obtain the loss of head due to friction by equation (10) or (11), p. 139, the diameter being known.

To the reduced level of the pipe at F, add the pressure which is required there in the main, and then add the loss by friction as above calculated, and the result will be the pressure above datum required at E.

Treat the pipe E J in the same way. If the requisite pressure at

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E work out less than before, no notice need be taken of this, for inasmuch as we shall arrange for a velocity in the pipe D E sufficient to feed both E F and E J, the pressure required for the length E F will be more than sufficient for E J.

Thus, in such cases where there is a branch, the pressure at the junction is found from that *one* branch for which it is a maximum.

The length E D is now treated in the same way. It has to carry enough to supply E F and E J. Hence the velocity is known. Calculate the loss of head; add this to the pressure at E already found, and we get the required head above datum at D.

Proceeding in this way, we can arrive at the necessary head, or elevation at the reservoir, to force the water through the system at the required rate, and thus estimate the suitability of the proposed scheme of pipes.

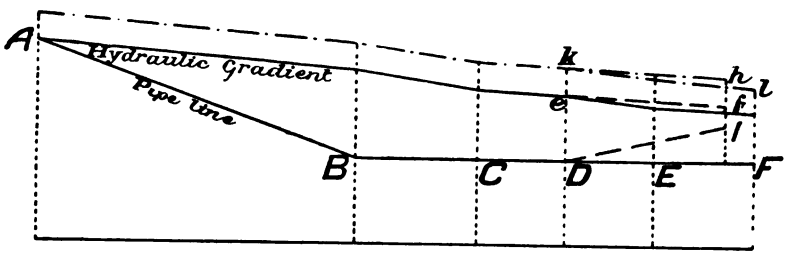

FIG. 60.

The pressures, as thus calculated, are above datum. If they be set off to scale, the resultant line is the hydraulic gradient.

If the levels of the pipe be set off on the same scale, the difference between the two lines tells the pressure on the pipe at each point.

Variation in Velocity.—The calculations just described assume that the velocity remains uniform throughout any pipe such as E F, fig. 60.

As a matter of fact this is not so, as water is being drawn off through the service pipes all the way along, hence the velocity decreases from E to F.

In Prof. Unwin's *Hydromechanics* it is shown that if we put

l = length of pipe,
d = diameter of pipe,
Q_0 = quantity of water entering a pipe,
Q_1 = quantity of water discharged through service pipes at regular intervals, so that

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$Q_0 - Q_1$ = quantity drawn off at the end of the pipe, then the loss of head by friction is approximately

$$h = \frac{64lz}{\pi^2 d^5 . 2g}\{Q_0^2 - Q_0Q_1 + \tfrac{1}{3}Q_1^2\},$$

and this formula should be used for exact calculations in such cases.

Or it may be taken that the loss of head is the same as would take place in the same pipe with a uniform discharge of $Q_0 - \cdot 45Q_1$.

Changing Diameters.—Now suppose it be found that the head required at D to force the water up the hill to I is greater than that required to force it towards F.

Then we may proceed in several ways:—(1) By raising the pressure at D.

Thus, referring to fig. 60, suppose we work backwards from F, as described, and obtain D*e* as the necessary head at D.

Let *ef* represent the hydraulic gradient along the pipe D I. Then I*f* will be the head remaining at I.

If this be insufficient, make I*h* equal to the necessary head, and if the other pipes remain as before, D*k* must now be the head at D, and the new hydraulic gradient *kl* will be parallel to the original gradient, but at a higher level.

This will increase the pressure throughout the system, all diameters remaining unaltered, and will necessitate a greater head at A. By putting in larger mains from A to D, the head at A may be sufficient.

(2) It may be possible, and cheaper, to increase the diameter between D and I, and so reduce the loss of head there.

(3) By resorting to artificial methods of raising the head at I, as by pumping.

Thickness of Mains.—The thickness required for the main is really settled, in most cases, not by the internal pressure on the main, but by that which is necessary to withstand shocks, vibrations, etc., mainly from outside.

So far as the internal pressure goes, if

t = thickness in inches,
p = pressure in lbs. per square inch,
r = radius of main,
s = tensile stress in lbs. per square inch that the material can bear in safety,

then $t = \frac{p.r}{s}$.

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Professor Burton recommends the following:—

$$t = \frac{(p+100)r}{s} + \cdot 25.$$

The term 100 in the bracket is introduced to cover the effect of the 'ramming' which takes place when the velocity is suddenly checked, as by the sudden closing of a valve.

s may be taken as about 3500.

The formula is not entirely satisfactory, for the effect of 'ramming' necessarily depends upon the diameter of the pipe and the velocity of flow, and hence a constant allowance, for it is hardly in accordance with the facts; but it is impossible, and it would be useless to attempt, to get an exact formula for all cases, and this formula is probably all that is required.

The allowance ·25 is for contingencies, irregularities in casting, etc.

Rankine's Rule.—Rankine recommended that the thickness

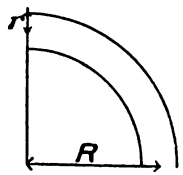

FIG. 61.

should never be less than $\sqrt{\frac{\text{diameter}}{48}}$, the diameter and the thickness being both in inches.

Testing Thickness.—For testing the thickness, pipe callipers may be used, or the weight may be calculated per foot length of pipe, and several of the pipes weighed.

Other Losses of Head.—(1) For a cylindrical pipe leaving a reservoir, without a bell-mouthed end piece, loss of head $= \cdot 505 \frac{v^2}{2g}$ (Unwin).

(2) If there is a bell-mouthed entrance, the loss of head is scarcely anything (Trantwine).

Bends.—(3) For a circular bend (fig. 61) the loss of head is—

$$\left\{ \cdot 131 + 1 \cdot 847 \left(\frac{r}{R}\right)^{\frac{7}{2}} \right\} \times \frac{v^2}{2g} \times \frac{\text{angle of bend in degrees}}{180},$$

where r = internal radius of pipe.

R = radius of bend of axis, other letters being used as before.

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In Trantwine's and other pocket books, tables calculated from this formula will be found.

Elbows.—(4) For an elbow (fig. 62)

$$h = 0{\cdot}9457 \sin^2\frac{\theta}{2} + 2{\cdot}047 \sin^4\frac{\theta}{2},$$

where θ = angle of elbow.

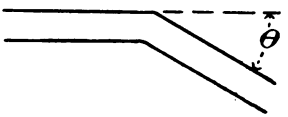

FIG. 62.

Reducing Pieces.—(5) For a reducing piece (fig. 63)

$$h = \frac{(k^4-1)}{2(k-1)} \cdot \frac{v^2 l}{c^2 r_2}.$$

where $k = \frac{r_1}{r_2}$

c = the value of $\sqrt{\frac{2g}{z}}$ in equation (14) p. 139,

and v = velocity of flow at larger end.

In the investigation on which this formula is based, c has been regarded as constant, and may be taken as the mean value for the diameters at the ends (p. 145), without sensible error.

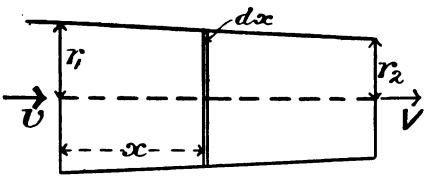

FIG. 63.

If V be put for the velocity in the pipe at the smaller end,

$$h = \frac{k^4-1}{2k^4(k-1)} \cdot \frac{v^2 l}{c^2 r_2}.$$

This is probably more convenient in most cases, as the discharge through the reducing piece will be the same as in the small pipe. If it be desired to allow for the reducing piece by adding an equivalent length to the small pipe, this length is given by the formula—

$$l_1 = l \cdot \frac{k^4-1}{4k^4(k-1)},$$ where l is the actual length of the reducing piece, and $k = \frac{r_1}{r_2}$ as before.

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The following table gives the values of $\frac{k^4-1}{4k^4(k-1)}$, for different values of k.

k	$\frac{12}{11}$ $=1{\cdot}09$	$\frac{11}{10}$ $=1{\cdot}10$	$\frac{10}{9}$ $=1{\cdot}11$	$\frac{9}{8}$ $=1{\cdot}125$	$\frac{8}{7}$ $=1{\cdot}14$	$\frac{7}{6}$ $=1{\cdot}17$	$\frac{6}{5}$ $=1{\cdot}20$	$\frac{5}{4}$ $=1{\cdot}25$	$\frac{4}{3}$ $=1{\cdot}33$	$\frac{3}{2}$ $=1{\cdot}50$	$\frac{2}{1}$ $=2{\cdot}0$
$\frac{k^4-1}{4k^4(k-1)}$	·81	·79	·77	·75	·72	·69	·65	·59	·51	·40	·23

Thus a reducing piece for changing the diameter of a main from 6 inches to 4 inches, and carrying the same discharge as the 4-inch main, will result in the same loss of head by friction as ·4 of its length of the 4-inch main ($k=\frac{6}{4}=\frac{3}{2}$ in this case).

Reduced to corresponding lengths of the larger pipe, the discharge being the same, we get the following table:—

$\frac{r_1}{r_2}$	$\frac{12}{11}$	$\frac{11}{10}$	$\frac{10}{9}$	$\frac{8}{7}$	$\frac{7}{6}$	$\frac{6}{5}$	$\frac{5}{4}$	$\frac{4}{3}$	$\frac{3}{2}$	$\frac{2}{1}$
$\frac{l_1}{l}$	1·14	1·16	1·48	1·23	1·28	1·34	1·44	1·62	2·03	3·75

Where r_1, r_2 are the radii at the ends, and l and l_1 are the length of the reducing piece and the equivalent length of the larger pipe respectively.

The proof of the above rule is as follows, referring to fig. 63:—

Take a strip dx at distance x from the larger end.

$$\text{Its radius}=\frac{r_1(l-x)+r_2x}{l}.$$

$$\text{Vel. of flow}=v.\frac{r_1^2l^2}{\{r_1(l-x)+r_2x\}^2}=v_x, \text{ say.}$$

By equation (14), $v_x=c\sqrt{mi}$, where i = rate of loss of head,

$$=\frac{dh}{dx}.$$

$$\text{and } m=\frac{radius}{2}.$$

$$\therefore\quad c^2.mi=v^2\frac{r_1^4l^4}{\{r_1(l-x)+r_2x\}^4},$$

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$$\text{or} \quad \frac{dh}{dx} = \frac{2.v^2 r_1^4 l^5}{c^2} \cdot \frac{1}{\{r_1(l-x)+r_2x\}^5}, \text{ putting } m = \frac{r_1(l-x)+r_2x}{2l},$$

$$\therefore \quad dh = \frac{2v^2 r_1^4 l^5}{c^2} \cdot \frac{dx}{\{r_1(l-x)+r_2x\}^5},$$

Integrating, $h = \frac{2v^2 r_1^4 l^5}{c^2} \quad \frac{\{r_1(l-x)+r_2x\}^{-4}}{4(r_1-r_2)}$.

Substituting the limits $x=l$ and $x=0$,

$$h = \frac{2v^2 r_1^4 l^5}{4.c^2(r_1-r_2)} \left\{ \frac{1}{(r_2 l)^4} - \frac{1}{(r_1 l)^4} \right\}$$

$$= \frac{v^2 l}{2c^2(r_1-r_2)} \quad \frac{r_1^4 - r_2^4}{r_2^4}$$

$$= \frac{v^2 l}{2c^2(k-1)r_2} \cdot (k^4-1), \text{ as above.}$$

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CHAPTER IX

DISTRIBUTION SYSTEMS

Cisterns.—In arranging a distribution system, there is considerable difference of opinion as to whether house cisterns should be allowed or not.

There is no doubt that it is better to provide house cisterns, but they should be kept clean. In view of the foul state into which they are too frequently allowed to get, however, some authorities think they should be avoided, and a constant service direct from the main provided.

The objections to this system are the great inconvenience which arises when a main has to be closed, even for a short time, for repairs, etc., the greater pressure on the taps and other fittings, and the necessity for providing rather larger branch mains, in order that if all the taps be running at once, the houses farthest from the mains may still have a supply.

Fires.—The provision necessary for the extinction of fires in towns is also a matter of some difference of opinion. Professor Burton recommends a minimum allowance of 200 cubic feet per minute in addition to the ordinary supply.

This is about equivalent to the supply for ordinary purposes to a town of 72,000 inhabitants at the rate of 25 gallons per head per day, and would require a 14-inch main to carry it alone, at a gradient of 1 in 200.

The fire hydrants are generally placed at a maximum distance of from fifty to a hundred yards apart.

No hydrant should be put on a smaller main than 4-inch diameter, and the diameters should increase about proportionately to the two-fifth power of the number of hydrants on it, or fed from it, and likely to be used at once.

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If mains are supplied at both ends, two hydrants may be placed on a 4-inch main.

Street Watering.—Hydrants for street watering are placed as shown in Plate IV.

General Arrangement of Pipes.—The two extreme arrangements of pipes are exemplified in figs. 64 and 65. In the one case (fig. 64) the pipes are all joined to another pipe at *one end only*, that being the end through which they are fed. The other end of each pipe is unconnected with any other. It forms what is known as a 'dead' or 'blank' end.

These are objectionable, as tending to cause stagnation, there being very little circulation at the end of the pipe.

The advantages of this system are that it is easy to locate waste, and that only one valve is necessary to cut off any district. If a leak is suspected, then if it be detected in the pipe *k*, it is only necessary

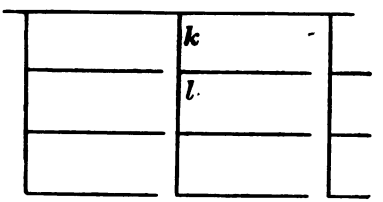

FIG. 64.

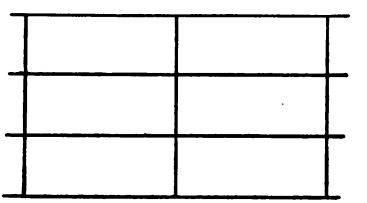

FIG. 65.

to follow up that pipe, and by trying at *l* and so on, the leakage can be found. Having located it, we need only close down one sluice valve, say at *k* or *l*, and the district where the leak exists is cut off, and the leak can be repaired.

These advantages are generally considered to be overbalanced by the objections already mentioned, and also by the further objection that a supply to any pipe can be drawn only from one end.

In fig. 65 all the pipes are shown running into, and joined with, one another.

By this means, the opening of a tap in any part of the system will cause the water to flow towards that point from all other parts of the system.

Thus complete circulation is everywhere ensured.

The disadvantages are that it is difficult to locate a leak, and a great many valves are necessary to enable any pipe containing a leak to be cut off for repairs.

In practice, no definite system applicable to all cases can be

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laid down. It is probable that no two engineers would give exactly the same arrangement of pipes in any case. But generally it is desirable to aim at something in between the two cases already considered, providing as far as possible:—

(1) That there shall be an efficient circulation at all parts of the system, each pipe being preferably supplied from both ends, and blank ends avoided.

(2) That the town be divided into districts of a moderate size, each connected preferably at two points with adjacent districts, so that by testing at those two points leaks may be located; and by closing valves at those two points the district may be cut off without interfering with the supply to the rest.

(3) That the fire hydrants will be on mains of sufficient size.

Thus, in fig. 66, an arrangement of loops is shown, which has these advantages. If a valve at *b* be closed, and if by means of a

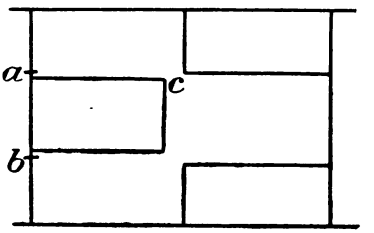

FIG. 66.

sounding rod or water meter at *a* it be found that there is more water flowing than should be the case, then a leak is located in the district *a b c*. This can then be cut off, and the leak repaired.

Blank Ends. Hydrants.—Where blank ends are unavoidable, which is often the case in places where future extension is likely, it is usual to place a street hydrant at the end of the pipe, by means of which it can be flushed out from time to time.

It is not intended here to enter into the construction of hydrants, as they are to be found in the catalogues of all makers of waterworks appliances.

Prevention of Waste.—The most efficient system of waste detection yet devised is that of dividing the system into districts as described, so that the supply to any district can be made, by closing certain valves, to flow through one known pipe; thus, in the system last described, the supply to the district *a b c* can be made to enter at *a* by closing the valve *b*.

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A bye-pass carrying a waste-water meter is placed on the main at that point. During the hours of least demand, say in the hours following midnight, the main valve is closed down, and the meter registers the quantity flowing into the district.

If this be excessive, there is probably waste going on.

If the district be subdivided, similar tests may be applied to the different divisions, and finally the service pipe valves leading to the different houses are tested by placing the valve key on the spindle and applying the ear to it. The flow can thus be heard. On a leak being located in this way, if on the main, it can be readily repaired; or if in the house fittings, the occupiers can be compelled to have them repaired.

Diagrams and particulars of waste-water meters and the mode of fitting them will be found in the makers' catalogues.

Scraping Water Mains.—For removing excessive incrustations on mains, water-scrapers are also made, and descriptions are given in the catalogues. They are forced along by the pressure of the water, generally, but are provided with a ring at each end for the attachment of a wire to remove them in case of accident.

They are liable to get jammed at bends.

A typical arrangement of distributing mains for a small town is shown on Plate IV.

The positions of the pipes, hydrants, and sluice valves are so clearly indicated that description is unnecessary.

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CHAPTER X

PUMPING MACHINERY

It is not within the scope of this book to give a comprehensive treatise on pumping machinery or on the machinery which may be advantageously employed in advancing the science of waterworks engineering, but a few words on these important subjects will not be out of place.

Where pumping must be resorted to, as is frequently the case both when the water is originally collected by gravitation, as well as when it is drawn from wells, the engineer must carefully consider not only the first cost of the pumping plant, which comprises the boilers, engines and pumps themselves, but also their foundations and the buildings in which they are to be placed, as well as the working cost, including wages, fuel, lubricants, lighting, interest on capital cost, sinking fund to pay off the same, and a provision for repairs and renewals. If the several items of yearly expenditure be capitalized and added to the capital cost of each installation of buildings and machinery, they can then be compared on an equal basis, and the arrangement which is most effective and at the same time economical can be selected.

Where the pumping main is long the velocity to be attained forms an important item of economical working, and should be carefully considered, as a comparatively small increase in the diameter of the pipe will involve a considerable increase in expenditure, while any diminution will increase the frictional resistance, put an increased pressure on the pipe, and will also involve the use of engines of greater power. Where the pumping main is but short, a velocity of 3 feet per second will probably be the most economical, but with long mains, greater velocities will, in most cases, secure better results.

No great refinements are possible, because pipes, except such as

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are of quite unusual dimensions, are made to stock sizes, and diameters increase by an inch or more at a time.

If the most effective, and at the same time economical, working is to be secured, each case must be considered on its own merits; but a few general remarks will enable the student to make an initial and general selection before going into more detailed calculations, and will point to those systems of working which may almost certainly be discarded.

If the lift be small, the quantity of water to be lifted large, and the amount regular, a centrifugal pump will, especially if designed for the work, be the cheapest in first cost, and the most economical in working, taking all the charges together. As the pumping is to be continuous, it is worth while to employ a highly economical engine; but whether a gas, oil, or steam engine should be employed is a matter for further inquiry.

Should the quantity of water to be lifted be subject to considerable variation in quantity, or of lift, the conditions are entirely altered. Variation in quantity may possibly be met by employing more than one pump, and adhering to the centrifugal type; but for continuous working with varying lift the centrifugal pump is inapplicable, unless it be possible to pump over a given level, and to reduce the head pumped against to something like a normal pressure by syphonic action after the water passes the highest point of lift.

On the other hand, it must be observed that the pumping out of dry docks, when the lift is very variable, but the quantity of water to be removed in a given short time is large, is almost invariably effected by centrifugal pumps, it being found that the pumps being in action for only a small fraction of each year, it is economical to sacrifice efficiency to saving in first cost, the items for wages remaining the same, the charges for interest, sinking fund, and probably repairs and renewals, being reduced, while the cost for fuel forms so small a part of the whole as to allow of considerable extravagance in this respect.

Centrifugal pumps as generally constructed are not economical machines, their efficiency being not greater than 58 per cent.; but as turbines are constructed to have efficiencies of from 75 to 78 per cent., there does not seem to be any satisfactory reason why a carefully designed centrifugal pump should not be equally efficient.

Where pumping is continuous day and night at a fairly regular

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quantity, and at a constant pressure, as in the case of a main supply to a large service reservoir, the best machinery will be the most economical; and it is probable that the steam engine will be found more efficient and economical than any other type of motor, though as gas engines are now constructed of great power, they run them very closely; as though the initial cost of the machine itself is great, the cost for foundations and buildings is less, and the cost of the producer plant for the gas engine is less than the corresponding expense with steam as the motive power.

To define, with even an approximation to accuracy, what type of steam engine is to be preferred under the conditions quoted, is so much a matter of opinion, that it is not possible to give a definite decision, especially as questions of locality affecting the cost of foundations, etc., may, unless included in the estimate, upset the calculation; but triple expansion rotative engines, built on the vertical inverted marine pattern, give good results. The same may, however, be said of other rotative engines, and some engineers still pin their faith on the old Cornish engine, notwithstanding the loss of steam occasioned by excessive clearance in the cylinder.

When the quantity of water to be pumped is large and the lift is variable, especially if the height to be pumped to be small, great economy cannot be secured; and if to these factors be added non-continuity in working, economy in first cost will in all probability more than counterbalance some loss in efficiency; and should the centrifugal pump be inapplicable, as is most likely, a direct acting triple-expansion steam engine of the Worthington type will best meet the requirements.

In small waterworks other considerations besides those of economy in first cost and of working come into play, as the item of wages forms an increasingly important factor with every reduction in size. Thus, say with a town of 4000 inhabitants, the quantity of water to be distributed daily is probably 80,000 gallons, or 8000 gallons an hour, pumping ten hours a day. Suppose the effective lift to be 70 feet, then the h.-p. required will be about 4½. A steam-engine of this size is never economical, and will probably use at least 6 lbs. of coal per indicated h.-p. per hour; the actual pumping hours are ten, but the working hours are at least twelve, while the boiler fires must be banked at night, causing a further loss of, say, 1 lb. coal per indicated h.-p. It is difficult for one man to give efficient attention to both boiler and engine, so that probably a man and a boy will be employed,

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whose combined wages are £115 a year. The cost per annum will work out something as follows:—

Interest and sinking fund on cost of engine, boiler, pumps, and buildings,	£43	2	0
Wages,	115	0	0
Coal,	55	0	0
Repairs and renewals, 7 per cent. on machinery, and 2½ per cent. on buildings,	35	18	0
Total annual cost,	£249	0	0

If in place of a steam engine a gas engine be used, there is no banking of the boiler at night; the engine can be started in five minutes after the arrival of the man, so that ten hours' pumping means ten hours' work, leaving repairs, which will probably be greater for the steam than the gas engine, out of consideration; there is no boiler, only gas-producing plant to look after. One man, at a smaller wage, can therefore do the work, the cost being as under:—

Interest and sinking fund on cost of gas plant, engine, pumps, and buildings,	£53	15	0
Wages,	67	10	0
Coke,	21	0	0
Repairs and renewals at 7 per cent. on machines, and 2½ per cent. on buildings,	51	9	0
	£193	14	0

The following figures are taken from actual practice, and represent the cost of working with steam as compared with household gas, the supply being very similar to that instanced above.

STEAM PLANT.

Interest and sinking fund on machinery and buildings,	£43	2	0
Wages,	144	6	0
Coals,	189	0	0
Repairs and renewals at 7 per cent. on machinery and 2½ per cent. on buildings,	35	18	0
Cost per annum,	£412	6	0

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Gas Plant.

Interest and sinking fund on machinery and buildings, .	£44	16	0
Gas at 3s. per 1000 cubic feet, ...	61	4	0
Wages, ...	71	10	0
Repairs and renewals at 7 per cent. on machinery and 2½ per cent. on buildings, ...	42	10	0
Cost per annum, ...	£220	0	0

It is assumed that in every case duplicate engines with duplicate and interchangeable pumps will be used, otherwise repairs must be made frequently and rapidly and therefore expensively; and in order to avoid actual failure of supply it is necessary to have a much larger service reservoir (one capable of containing something like a week's supply), than would be necessary if duplicate machinery were provided.

Oil engines, which require no producer plant and for which the fuel occupies little space, the transport being inexpensive, may supersede gas engines, especially where carting of fuel is an important matter, but it is believed that no oil engine in the market can be said with certainty to be as economical as gas engines of well known and recognised efficiency.

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CHAPTER XI

REQUIREMENTS IN CONNECTION WITH WATERWORKS

It should, it is considered, be the aim of every waterworks engineer to make himself thoroughly acquainted with the works under his superintendence, not only from their commercial and administrative aspect, but from a scientific point of view, yet the amount of information obtainable by the layman, or even by the engineer not directly connected with waterworks, on subjects which are of the first importance in satisfactory and economical laying out of undertakings for the supply of water, is exceedingly limited.

One of the first questions to be answered is the amount of the rainfall in the district under consideration, yet notwithstanding the labour of Professor Symons, his successor Mr Sowerby Wallis, and others, the information obtainable is far from perfect; there are but few of the rivers and streams in England which are accurately and systematically gauged. It is doubtful if the amount of water actually running into the many reservoirs which supply a large part of the population of the country is known except in a few isolated cases, nor is the supply delivered to the population known with any degree of certainty. The relations between rainfall and evaporation, the amount of water which is lost from the many causes which influence its absorption, are vague; and many other problems which might be in a fair way to be solved, were proper records taken and published, remain unsettled. Interesting observations have been made by private individuals on the percolation of water through different soils and under different conditions, yet these also are unpublished. Wells are numerous, but their statistics, their relation to one another, to the rainfall and the effect of seasonal change on the level of the water, are practically unknown.

The writer is strongly of opinion that the utmost publicity should be given to all the knowledge which exists on the subjects referred

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to; but the work, to be complete, should not be directly in the hands of private individuals, but of conservators of the several drainage areas into which the country should be divided for purposes of the prevention of pollution of rivers and streams, water supply, and drainage, whose duty should consist in safeguarding the waters under their authority from pollution, in utilizing them to the best advantage both from a sanitary and economical point of view, and in preventing, so far as their powers extend, the wasteful and unnecessary duplication of small sewage disposal works; and finally, they should not only be empowered, but be under obligation, to systematize the rainfall statistics within the area of each, to ascertain the flow of the main river under their charge and each of its tributaries, to keep a record of the changes of level in a number of representative wells, and to publish within a reasonable time the whole of the information so collected. The towns, in like manner, should be obliged to afford to the conservators full and accurate information as to the extent of their drainage area, the rainfall in it, the water collected from it, the water supplied to the town, and its uses.

THE END

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